GEORGIA DEPARTMENT OF TRANSPORTATION

PAVEMENT DESIGN MANUAL

Pavement Design Manual Mission Statement

To provide a formal, uniform, and comprehensive process, and serve as a source of information that fosters thoughtful engineering in the design of pavements.

Managed and Maintained By

Developed With Cooperative Effort By Offices Of

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Keeping Georgia on the Move

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Revisions

Date	Chapter Name	Pages Affected	Approved By	Comments

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Foreword

This manual provides guidance for developing the history and necessary information that may be needed in designing both a rigid and flexible pavement structure. Most users of this manual may only need limited chapters. A summary and guidance for use of this manual is as follows:

- Chapter 1 – Mission Statement, revisions, history
- Chapters 2, 4, 5, 6 – Geotechnical, pavement layers, subgrades
- Chapters 3, 7, 8 – Geotechnical, understanding pavement design parameters, loads, and stresses
- Chapters 8 and 9, Appendix C – Geotechnical, existing pavement evaluation
- Chapter 10 – Pavement type selection, the decision process for selecting rigid or flexible pavement
- Chapter 11 – Hands-on rigid and flexible design process. For the experienced designers wanting to do a quick design, refer to the following sections of this chapter: asphalt, Chapter 11.4.1; PCC jointed, Chapter 11.5.1; CRC, Chapter 11.5.2; minor projects, Chapter 11.6.3; Ramps, special designs, Chapter 11.6.1
- Chapter 12 – Pavement maintenance
- Chapter 13 – Pavement design examples and flow chart linking Chapters to the overall selection, design, and approval process

To be able to make informed decisions from concept to approved design, the user is encouraged to read through and be familiar with this manual.

1 Introduction

1.1 Mission Statement

To provide a formal, uniform, and comprehensive process, and serve as a source of information that fosters thoughtful engineering in the design of pavements.

1.2 Abstract

Within this manual, the reader will find a wealth of information that will provide a foundation for the design of pavement structures. This foundation consist of a historical perspective on pavement design in Georgia, basic pavement structure terminology and principles, construction principles, geotechnical considerations, bases, pavement distresses, necessary information needed in the evaluation/ design of pavement (and where to get it), design of pavement, how to make a good design decision based on the analysis, and ability to project far into the future in order to adequately consider maintenance concerns.

Education in pavement concepts and principles, gathering of information, analysis of data, recommendations, and implementation summarize the scope of this manual.

Pavement materials are high dollar items and they make up a substantial portion of the project costs in most jobs. The pavement costs can vary from a low of 8% of total costs in an interchange reconstruction project involving major bridge work, to 25%-43% of a standard four-lane widening/reconstruction project, and up to 80% of the total costs for a maintenance project involving mostly overlay and shoulder work. Good engineering judgment and thoughtful consideration of the design will provide a cost effective and structurally sound end product.

1.3 Process to Revise Manual

1.3.1 General

Revisions to this Pavement Design Manual will be necessary as new technology and research reveal a greater understanding of paving materials, their characteristics, placement techniques, and maintenance criteria. Procedural changes by the Department will also necessitate the revision of this manual. Revisions must be implemented in an orderly and consistent fashion. All revisions must address the electronic version of the text (located at - TBD), and the best method to disseminate/ implement the information via email, website alerts, training, and so on.

Revisions to the manual may be proposed by anyone (GDOT, local governments, municipalities, consultants, etc.). Proposed revisions shall be sponsored and presented by a Pavement Design Committee Member (PDCM) to the Pavement Design Committee (PDC) (See TOPPS 5560-1 and 5560-2 (Chapters 1.3.2 and 13.1.1)). The requested revision will be discussed in the quarterly PDC meeting. The chairman of the PDC shall determine whether the request is in the best interest of the Department. The chairman may choose to discuss the matter in the meeting if a speedy decision is warranted, or may choose to assign the request to one or several others in a sub-committee to review the request and provide a recommendation. In either case, the method and schedule shall be determined by the chairman. Upon satisfactory review (as determined by the chairman), the committee shall vote on the request (in whole or in part as deemed appropriate).

1.3.2 Revision to the Electronic Text

Revisions to the electronic text must include a new version date. The chairman shall determine whether the section, sub-section, or sub-sub-section shall be replaced in its entirety. Revisions shall follow the standard formatting used throughout the entire document.

1.4 Pavement Design Committee Members

PDC Members, Bylaws, and pavement design submittal guidance can be found on the Department's web site at http://www.dot.state.ga.us/topps/index.shtml. TOPPS 5560-1 and 5560-2 should be routinely checked for revisions.

Excerpt From TOPPS 5560-1, Pavement Design Committee Members

The following Department and FWHA personnel are to constitute a Committee on Roadway Pavement Structures:

- State Pavement Engineer (Chairman)
- State Pavement Design Engineer (Secretary)
- Construction Office Representative
- Maintenance Office Representative
- Road and Airport Design Representative
- Urban Design Representative
- Engineering Services Representative
- FHWA Representative
- Consultant Design and Program Delivery Representative

The purpose of the Committee will be to develop criteria for the determination of pavement structure component types and to approve the pavement structure design on all projects presented by the design offices for consideration. No subsequent changes in the design shall be made without the approval of the Committee.

The Consultant Design, Road and Airport Design, and Urban Design Engineers will be responsible for making the analysis of proposed pavement structures for projects within their design jurisdiction. This structural analysis will be based on factors furnished by both the Office of Materials and Research and the Office of Planning, and approved design criteria for both flexible and rigid pavement structures.

The State Construction Engineer will advise the Committee as to how current construction procedures will affect the pavement structure, and as to what construction difficulties might be inherent in any proposed structure and how the structure might be modified to counter those difficulties.

The Construction Office Representative will advise the Committee as to how current construction procedures will affect the pavement structure, and as to what construction difficulties might be inherent in any proposed structure and how the structure might be modified to counter those difficulties.

The Maintenance Office Representative will advise the Committee as to the service of previously constructed pavement structures, and will make recommendations as to how these structures might be modified to counter poor service history.

A proposed design will be presented to the Committee by the appropriate design office. Any member of the Committee may make a motion to modify the proposed design; however, such motions should be supported by explanation as to why the change is desirable and what they consider to be a reasonable alternative to the proposed design. After a second and discussion, the Chairman shall call for a vote on the motion.

After approval of a pavement design by the Committee the appropriate design office will make any necessary corrections and then forward it to the State Pavement Engineer for final approval. An Approved Pavement Design shall carry the signatures of the person who prepared the pavement design analysis, the appropriate design office engineer and the State Pavement Engineer.

1.5 History of Pavement Design

The first road builders of any significance were the Romans, who saw the ability to move quickly as essential for both military and civil reasons. The earliest examples of Roman road building date back to 312 B.C. It is from the Romans that the term highway comes, as all their roads were elevated 1m above the local level of the land. This was to minimize the risk of an ambush. The design standards set by the Romans in terms of durability far exceeded anything achieved after the fall of the empire until modern times.

"Via Appia," or the Appian Way leading to the modern city of Rome is a testament to the durability of Roman road construction. It remains in existence until today.

The hyperlink below has a collection of photographs taken along the "Via Appia" in modern times.

http://www2.siba.fi/~kkoskim//rooma/pages/VAPPIA.HTM

The following hyperlink is for a "milestone" along the Via Apia.

http://www2.siba.fi/~kkoskim//rooma/pages/171_020B.HTM

Romans also classified their roads in terms of importance. The public roads, or "viae publicae," were of the highest order of importance, were up to 40 feet wide. The least important were private roads or "viae privatae," which were built and maintained by the landowner.

The Roman approach to road design is essentially the same as that in current use in the fact that the roads were constructed of several different layers, increasing in strength from the bottom to the top. The lowest layer was normally rubble; intermediate layers were made of lime bound concrete and the upper layer was a slag or lime grouted stone slabs. The thickness of the layers was varied according to the local ground conditions. The link below is to a brief article on Roman road building.

http://www.battleoffulford.org.uk/ev_roman_rd_constrct.htm

There was surprisingly little innovation in the field of pavement design from the Roman times until the mid 1950's AASHO Road Test.

1.6 Modern Pavement Design

Recognizing the need to understand the relationship between pavement deterioration and axle load repetitions, for the proper taxation of trucks, the AASHO Committee on Highway Transport authorized the AASHO Road Test in 1951. In 1956 construction of six test loops began in Ottawa, Illinois. In November of 1960 traffic ended on the test facility and the resulting information gained from the testing and research effort was used in the Interim Pavement Design Guide published in May 1962. This effort was preceded by the smaller, less robust experiments of Road Test One in Maryland and the Western Association of State Highway Officials (WASHO) Road Test.

The resulting iterations of the original design guide are listed along with the major improvements.

Date	Publication	Major Advancement
1961	Interim Guide for the Design of Rigid and Flexible Pavement Structures	Established a modern, consistent pavement design system
1972	AASHTO Interim Guide for Design of Pavement Structures	Added information based on subsequent research and experience.
1981	AASHTO Interim Guide for Design of Pavement Structures	Revision of the Portland Cement Concrete Pavement Design
1986	AASHTO Guide for Design of Pavement Structures	Guide Officially Adopted by AASHTO including a new section on rehabilitation
1993	AASHTO Guide for Design of Pavement Structures	Changes to the Overlay Design Procedure and the addition of 14 new design considerations
1998	Supplement to the AASHTO Guide for Design of Pavement Structures	Improvement to the Rigid Pavement Design performance models

TABLE 1.1 – ASHTO PUBLICATIONS

It is important to note that the performance equations used in all iterations of the AASHTO Pavement Design Guide were developed for the specific conditions of the AASHO Road Test. These equations have some significant limitations:

- The equations were developed based on the specific pavement materials and roadbed soil present at the AASHO Road Test.
- The equations were developed based on the environment at the AASHO Road Test only.
- The equations are based on an accelerated two-year testing period rather than a longer, more typical 20+ year pavement life. Therefore, environmental factors were difficult if not impossible to extrapolate out to a longer period.

The loads used to develop the equations were operating vehicles with identical axle loads and configurations, as opposed to mixed traffic.

In order to apply the equations developed as a result of the AASHO Road Test, some basic assumptions are needed:

- The characterization of subgrade support may be extended to other subgrade soils by an abstract soil support scale.
- Loading can be applied to mixed traffic by use of ESALs.
- Material characterizations may be applied to other surfaces, bases, and sub-bases by assigning appropriate layer coefficients.

The accelerated testing done at the AASHO Road Test (2-year period) can be extended to a longer design period.

When using the 1993 AASHTO Guide empirical equation or any other empirical equation, it is extremely important to know the equation's limitations and basic assumptions. Otherwise, it is quite easy to use an equation with conditions and materials for which it was never intended. This can lead to invalid results at the least and incorrect results at the worst.

1.7 Mechanistic-Empirical Pavement Design

In an attempt to address some of the limitations of the empirically based design procedures of the AASHTO Guide for Design of Pavement Structures, the NCHRP 1-37 research project was initiated in 1998.

The overall objective of the project called for the development of a guide that used existing mechanistic-based models and databases reflecting current state-of-the-art pavement design procedures. The products of this research include the following:

- The Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures
- User-Oriented computational software and documentation based on the Design Guide procedure.
- The Mechanistic-Empirical Design Guide, released in March 2004, represents a major change in the way pavement design is performed. The Design Guide requires and considers site-specific inputs of traffic, climate, subgrade, and existing pavement condition. The Mechanistic approach provides a framework for continuous improvement and allows the designer to consider changes in trucking, materials, construction, and design concepts in a way that is impossible with the empirical based design procedures that were extrapolated from the traffic loading and climatic conditions of Ottawa, Illinois.

It is expected that AASHTO will adopt the Mechanistic-Empirical Design Guide as a replacement or supplement to the 1993 AASHTO Design Guide for Pavement Structures. More information on this topic can be found in Chapter 11.2.5.

1.8 Basis of This Manual

The 1996 GDOT Pavement Design Manual is based on the 1972 AASHTO Interim Guide for Design of Pavement Structures with the revision of Chapter III in 1981 for the design of rigid pavements. The intent of this manual is to update the 1996 GDOT Pavement Design Manual to reflect the advancements in pavement design procedures and to supplement those procedures to reflect the unique material, climatic, and traffic conditions present in Georgia.

This manual is still based on the 1972 AASHTO Guide for the Design of Pavement Structures as stated above. Although the data collected and the relationships determined from the AASHO Road Test are limited by the scope of load, weather, and traffic conditions, the GDOT has had good success in applying these relationships to the design of highway pavements. This is evident by the overall condition of the highway pavements throughout the state.

References:

D. Margary, *Roman Roads in Britain* (2 vol., 1955–57; rev. ed. 1967).

AASHTO Interim Guide for Design of Pavements 1972. American Association of State Highway and Transportation Officials. Washington, D.C., 1974.

AASHTO Guide for Design of Pavements. American Association of State Highway and Transportation Officials. Washington, D.C., 1993.

Supplement to the AASHTO Guide for Design of Pavements. American Association of State Highway and Transportation Officials. Washington, D.C., 1998.

Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. National Cooperative Research Program. Transportation Research Board National Research Council. Washington D.C., March 2004.

Smith, K.D., Zimmerman, K.A., Finn, F.N. *The AASHO Road Test: Living Legacy for Highway Pavements*. TR News Number 232. Transportation Research Board. Washington, D.C. June 2004.

Georgia Department of Transportation

2 Pavement Structure Basics

2.1 Introduction to Layered Systems

Primarily heavier wheel loads, higher traffic volumes and the recognition of various independent distress modes contributing to pavement failure, such as rutting, shoving, and cracking, have brought about changes in design and construction of pavement systems. Part of the solution has been the introduction and use of stabilized base and subbase materials to increase rigidity of these elements of layered systems.

Flexible pavements are layered systems with the better materials on top. The sum of various layers cannot be represented as a homogeneous mass. Utilizing Burmister's layered theory is appropriate. Each layer is assumed to be homogeneous, isotropic, and linearly elastic with an elastic modulus E, and a Poisson's ratio ν. Each layer has a finite thickness except the lowest layer is infinitely thick, and all layers are infinite in the lateral directions. Full friction is assumed to develop between each layer at its interface. The load-carrying capacity of a flexible pavement is brought about by the load-distribution characteristics of the layered system. ("The Theory of Stresses and Displacements in Layered Systems", D.M. Burmister)

Rigid pavements are constructed of PCC (Portland Cement Concrete) and may or may not have a base course between the pavement and subgrade. The concrete pavement, because of its rigidity and relatively high modulus of elasticity, tends to distribute the applied load over a relatively wide area. The slab itself supplies the major portion of the structural capacity.

Resilient Modulus (M_R): Determination of the resilient modulus uses a dynamic response test where the strain used to calculate the modulus is the recoverable portion of the deformation response. Generally, the specimen is subjected to about 200 to 1000 "conditioning" repetitions, depending upon the material (sands, silts, clays, asphalt), and then M_R values can be calculated after an additional 150 to 200 repetitions at each stress state. For soils, approximate values for M_R can be interpolated from Soil Support Values; however, these values are usually significantly higher than triaxial test values in sands, silty sands, and clayey silty sands, i.e., the Piedmont and Coastal Plain.

- Most paving materials are not elastic but experience some permanent deformation after each load application; however, if the load is small relative to the strength of the material, the deformation under each load repetition is nearly completely recoverable and proportional to the load and can be considered elastic. Selecting high M_R values leads to increased susceptibility to thermal and fatigue cracking. Linear viscoelasticity is excluded from this discussion.

Specific layers that do not contribute to the structural strength of pavement systems are:

- PEM (Porous European Mix)
- OGFC (Open Graded Friction Course)
- Leveling Layer
- Micro Seal/Micro-Surface Treatment
- Chip Seal

2.2 Structural Layers in Flexible Pavements

The quality of the Hot Mix Asphalt (HMA), base course, subbase course and subgrade is indicated by the structural layer coefficients. Layer coefficients are a measure of the relative ability of a unit thickness (1 inch) of a given material to function as a structural component of the pavement. Structural Numbers (SN) are a function of layer thickness, layer coefficients, soil support values and drainage coefficients. Historically GDOT has not utilized drainage coefficients. Layer Coefficients, per inch, for HMA may be in the range of 0.3 to 0.44, untreated base course in the range of 0.12 to 0.16 depending upon materials utilized, and subbase in the range of 0.05 to 0.11 depending upon area of the state.

Note: See Appendix D for table of values for structural coefficients.

2.3 Structural Layers in Rigid Pavements

A base course can be utilized to reduce critical stresses in the concrete. Generally, it is uneconomical because the same critical stresses in the concrete slab can be obtained by small increases in the thickness of the slab. A base course would be utilized for the following reasons:

- Control of shrinkage and swelling of fat soils by controlling amounts of water entering the subgrade
- Expedite construction

With the presence of either or both of these conditions base courses are normally utilized with rigid pavements.

Base Layers

GDOT does not advocate drainable base courses; rather its present (2005) strategy is to minimize moisture under rigid pavements.

2.4 Flexible Pavement

Note: The project location determines base layer selection.

2.4.1 Graded Aggregate Base Layers

GAB can be placed in a single layer or multiple layers depending upon its thickness; layers not to exceed 8 inches and not to exceed 2 layers. Layer Coefficients may be in the range of 0.12 to 0.16

2.4.2 Cement Stabilized Layers

For heavily traveled pavements, the use of a cement-stabilized base course is common practice. M_R of cement-stabilized base is correlated with the unconfined compressive strength to obtain layer coefficients. Layer Coefficients may be in the range of 0.10 to 0.30. Cement stabilized graded aggregate and cement stabilized soil aggregate may be placed in two equal layers or one layer not exceeding 8 inches.

2.4.3 Asphaltic Base Layers

Full depth asphalt pavements are constructed by placing multiple layers of HMA directly on the subgrade or improved subbase

2.5 Rigid Pavement

Note: Project location determines base layer selection.

2.5.1 Graded Aggregate Base Layers

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2.5.3 Asphaltic Inter-layers

The use of asphaltic material for base is not typical in the design and construction of today's rigid pavements.

A 3-inch thick asphaltic interlayer is, however, designed and constructed as part of a rigid pavement. This interlayer serves the following purposes:

- As a layer that separates PCC Pavement from all underlying layers and pavement in an unbonded overlay design and construction
- As a drainage layer for surface moisture infiltration
- As a separator layer for subgrade fines from the PCC Pavement

- As a layer used in the Construction staging and convenience.
- In rigid pavement design and construction, asphaltic base is usually 3 to 5 inches. Its purpose is to separate fines of the subgrade from the Portland Cement Concrete (PCC), to keep moisture in the PCC, and construction convenience.

NOTE TO MANUAL USERS: The definition of "subbase" and "subgrade" differ somewhat within the industry. This manual will use the following distinctions for design purposes:

-If it is a subgrade, the material strength properties will be defined with a "Soil Support Value" (Flexible Pavement) or "Modulus of Subgrade Reaction"(Rigid Pavement).

-If it is a base or subbase, the material strength properties will be defined with a "Structural Number."

2.5.4 Subbase Layers

- Quite often subbase layers are utilized to effect economical solutions. Local aggregates (clayey sands, sandy gravels, clayey gravels, IIB3 or better) can be utilized to reduce the thickness of the more expensive base course.
- Subbase layers may be stabilized by utilizing cement or lime to improve layer coefficients.

2.6 Subgrades

Properly prepared subgrades play a critical role in pavement performance. The ability to maintain the subgrade at 100 % of the maximum dry density through proper drainage and control of infiltrating surface water is critical for the subgrade to provide reliable Soil Support Values.

Subgrades soils can be in-situ soils or select materials. This layer is considered to be infinitely thick in accordance with Burmister's layered theory although the Standard Specifications generally defines the subgrade as the top 12 inches of the roadbed.

Construction of the subgrade with in-situ soils consists of work on at the top 6 inches of the roadbed, which are scarified and compacted to 100% of the laboratory maximum dry density, according to GDT-7. For embankments construction, layers are placed for the full width of the cross-section in thickness not to exceed 8 inches (loose measurement) before being compacted to 95 % of the laboratory maximum dry density in the optimum moisture range.

Select materials would only be utilized in counties south of the Fall Line if at-grade in-situ soils were too poor to provide acceptable Soil Support Values for the pavement section.

Blending lower quality local materials with select materials can improve the Soil Support Value to design levels.

2.7 Pavement Layers - Function and Costs

2.7.1 Subgrade

The subgrade is the top layer of embankment below the pavement section (unless a subbase is used).

The most cost-efficient subgrade uses unaltered material from the project or from a borrow site close to the project. Class IIB3 and better soils are normally suitable for constructing the subgrade (GDOT Standard Specification 208.3.5.B.2.f.1). If Class IIB3 material is not available near the project the Soil Survey will normally recommend the addition of Stabilizer Material (GDOT Standard Specification 209.2.B).

Note: See Chapter 4 of this manual for more detailed information on subgrades and subgrade stabilization methods.

2.7.2 Subbase and Base

The subbase is the top layer of improved embankment below the pavement section.

The base is the bottom layer of the pavement section that is used in design.

Base material is normally “Graded Aggregate Base” (GAB) or “Asphaltic Concrete” but project specific conditions may allow Soil-Cement Construction, Sand-Bituminous Stabilized Base Course, Sand-Clay, or Chert, Soil Aggregate Construction, or Cement Stabilized Soil Aggregate base alternatives.

Note: See Chapter 5 for more detailed information on bases and subbases.

2.7.3 Asphaltic Concrete Pavement Section

An asphaltic concrete pavement section normally consists of GAB or asphaltic concrete base layer, and asphaltic concrete base, binder and surface layers. On some projects an asphaltic concrete riding surface is provided that is not considered as part of the structural pavement design.

Note: See Chapter 6 for detailed information on Asphaltic Concrete Pavement.

2.7.4 Portland Cement Pavement Section

A Portland cement concrete pavement section normally consists of a GAB base layer, an asphaltic concrete interlayer and a Portland cement slab. The slab will either be plain Portland cement concrete (jointed) or continuously reinforced concrete.

Note: See Chapter 6 for detailed information on Portland Cement Concrete Pavement.

2.7.5 Relative Costs

Many factors must be considered when selecting the most appropriate pavement section for a project. GDOT policies provide guidance on many of the selection criteria. The availability of material should also be considered (especially for selecting the subgrade, base and subbase. In addition to these other factors the cost of the pavement section should be considered. The availability of local material will affect the cost due to hauling costs and market conditions. It is difficult to determine what pavement section will be the most cost effective on a specific project. Normally a cost effective pavement design minimizes the thickness of a higher cost material by substituting more thickness of a less expensive material. However there are too many factors that are variable and unknown to determine the "best" section for any one project before the project is actually let to contract. For example, if an individual contractor has a large stockpile of recycled asphalt pavement (RAP) that contractor may be able to provide some additional thickness of asphaltic concrete at a lower cost than additional thickness of GAB. Another contractor may have equipment and work crews available to construct a soil-cement base. The project manager does not have access to this type of information.

The best method to determine the most cost effective pavement section is to design alternate paving, base and improved subgrade sections and include these in the project plans. This allows each contractor to pick the paving section that he can construct at the lowest cost.

3 Design and Construction Parameters

3.1 Design Parameters

3.1.1 Traffic

Traffic is one of the most influential design parameters. The makeup of traffic is of particular importance, especially the amount of trucks and the ratio of Single Unit (SU) to Multi-unit (MU) trucks. For rigid pavements, one 18K single axle load (ESAL) for a SU truck equates to 670 passenger car loadings. A MU truck equates to about 5.36 SU loadings. For flexible pavements, one 18K ESAL for a SU truck equates to 375 passenger car loadings.

Note: The Project Manager should take special care to ensure that the traffic counts for both the opening year and the design year are up to date.

3.1.2 Regional Factors

Weather and erosion related factors effect the design decisions in Georgia. The Department does consider the freeze-thaw depth; however, adverse weather-like hurricanes can affect Georgia’s coastal areas. Other regional factors such as erodable soils are important also. Analysis of historical data for weather and soil related influences have been investigated by the Department. From this analysis, Regional Factors were assigned that represent a composite influence. See Appendix H.

3.1.3 Soil Support

Soil Support, a region-specific value for the structural capacity of a soil, plays a significant role in designing pavements. It is based on the CBR (California Bearing Ratio). Currently GDOT uses soil support values that range from 2.0 to 4.5. The higher number represents a soil profile with greater strength. Your analysis may include historical research of similar projects in the region, field visits, soil borings, and/or lab tests and analysis. Along with the existing materials are the proposed materials. Knowing what you have to work with in the region often guides the options that may be employed in the design. Investigations of existing soils characteristics may allow simple use of those materials compacted according to specifications or may require that the soils be enhanced, with other natural or synthetic materials. See Appendix G.

3.1.4 Design Life

Historically, GDOT has considered the design life of flexible pavement and rigid pavement to be 20 years. The premise behind this timeframe was largely influenced by the maintenance of the structures. Historically, GDOT maintenance plans included the resurfacing of flexible pavements every seven years. Overlays would occur at year 7 and 14, and year 20 would signal the effective end of the pavement's life. Similarly, rigid pavement maintenance plans included resealing the joints every 10 years. However GDOT has observed that its existing rigid pavements lasted for 30 years or more with very little maintenance. The far more robust rigid designs that GDOT is considering should last considerably longer than 30 years. As a result, the Department is attempting to depart from the standard 20-year design life and to use Life Cycle Cost Analysis to estimate the cost effectiveness of these more robust rigid pavement designs.

3.1.5 Site Specific Consideration

Consider the site and the type of project. Differing projects may include:

- Intersections
- Bridges
- Rural Passing Lanes
- Urban or rural environments with associated typical sections
- Interstates

3.2 Construction Considerations

3.2.1 Maintenance of Traffic/Staging

Maintenance of Traffic (MOT) is a major consideration in achieving the desired quality pavement structure. The contractor must be able to build the proposed structure to specifications. Communication with construction liaisons and District Construction Engineers (DCE) is critical. The construction liaisons/DCE will enlighten the designer about the conditions in the project area of the proposed design. Staging strategies go hand in hand with the MOT considerations. The designer should communicate his recommendations with the construction liaison/DCE as to the best methods of staging the proposed pavement structure. Current research has indicated that the traveling public would rather endure a more intense and shorter staging effort than one that is drawn out. Indeed, fewer shifts in traffic staging are preferred.

3.2.2 Lift Thickness

Lift thicknesses are predominantly driven by the maximum stone size and the historical ability of contractors to place the material in an acceptable fashion. For surface layers this includes smoothness requirements from GDOT Standard Specification 400.3.06 Table 7. Designers will select lift thicknesses from GDOT Standard Specification 400.3.05 Table 5 and should consider the number of stages required for a particular project, the opportunity for traffic to be staged (and how long) on certain mix types, the ease of construction, and the ability to adjoin structures within the project limits, and/or tie to adjacent projects.

The binder and surface lift thicknesses are often driven by safety considerations. The maximum allowable drop-off between adjacent lanes cannot exceed two inches.

3.2.3 Milling Depth

In Georgia, milling is now a much more common procedure than in years past. This is largely due the technological advancement in milling a pavement to relatively tight specifications and a greater emphasis on the recycling of existing pavement materials. This reused material is called Recycled Asphalt Pavement (RAP). Typically, the milling depth of an existing pavement is recommended in an existing pavement evaluation. This milling depth is influenced by analysis of the pavement cores. Often the milling depth will be 1.5 inches. However, designers should consider additional milling depth to remove existing rutting, cracking, and/or other miscellaneous structural failures, and surface conditions.

A pavement evaluation will also include a proposed pavement structure recommendation. If a pavement is inlayed with the same depth of pavement removed via milling, there will be no increase in structural value. Most pavements are under-designed at their initial construction because overlay is expected to occur. The designer should consider adding additional structure to lengthen the service life of a pavement rehabilitated as part of the project.

3.2.4 Drainage of Pavement Surface

Significant strides have been made in the drainage of pavement structures. Most commonly known in Georgia would be the old “D” mix. This was a drainage layer that was placed on top of the surface course in order to address ponding in flat areas, and the “water spray” that commonly is associated with tractor trailers. The most common application was on the interstates in the Atlanta metro area. Over the last decade, this “conventional mix” has been replaced with the Open Graded Friction Course (OGFC) placed in a ¾ inch thick layer and most recently by the Porous European Mix (PEM) placed in a 1 ¼ inch thick layer. The pores in the pavement structure provide a path of least resistance for the water that is displaced under the wheels of vehicles.

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4 Subgrades

4.1 General

The ability of the subgrade (natural or improved underlying soils) to support loads transmitted from the pavement is a critical factor in pavement design. In roadway construction, the subgrade provides the foundation for the pavement. Different types of soils have different abilities to provide support. In general, a sandy soil, for example, will support greater loads without deformation than a silty clay soil. Thus, for any given traffic volume and weight of vehicles using the roadway, a greater pavement thickness must be specified on soils with lower subgrade strengths. Soils are classified for design purposes to predict subgrade performance based on either laboratory or in-situ field-testing. Within GA DOT, soil classification is based on the sieve analysis (laboratory test procedure GDT-4), the volume change (GDT-6), and the maximum dry density (GDT-7 or GDT-67) of the soils being tested.

4.2 Subgrade Strength Determination

The designer's first concern should be to evaluate the load supporting capacity that the project's soils are likely to provide under any conditions, except severe frost.

The basic design thickness of a flexible pavement structure required for a given class of service is dictated by the degree of soil support available. While the make-up of this design thickness depends upon the character of traffic expected and the quality of materials available, and to some extent upon climate, the total thickness needed to protect the subgrade soil from overstress remains the same for a given design situation.

The CBR (California Bearing Ratio) test is the one most widely used in the United States and many other parts of the world to evaluate the load carrying capacity of soils under normal (non-frost) conditions (ASTM D 1883 or AASHO T-193.

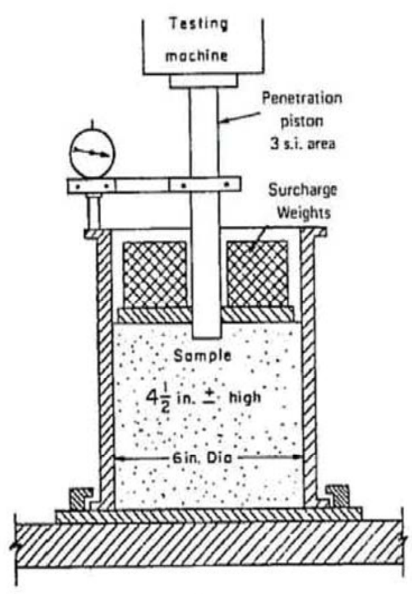


FIGURE 4.1 – CBR TEST APPARATUS

This empirical test measures the resistance of a compacted soil to the gradual penetration of a cylindrical piston about two inches in diameter, or specifically 3 square inches in end area. The CBR of a given soil is the ratio between (a) the load required to cause either 0.1 inch or 0.2 inch penetration of the piston into the soil being tested and (b) a standard load, either 1000 lbs. or 1500 lbs., respectively, expressed as a percentage (see Figure 4.1).

The test is usually performed in the laboratory; however, it may be performed on actual components of pavements in the field. For example the CBR test may be run on base, subbase, or subgrade material.

Note: The lab test on granular mixtures containing coarse particles or mixtures stabilized with cement or other admixtures may be influenced by dimensional effects so as to make the results unreliable.

There is some variation between the procedures used by different agencies to determine the CBR value for a soil in the laboratory. The GA DOT prepares test specimens from each sample at a range of molding moisture contents and three compactive efforts; the design CBR, selected from the resultant “family of curves,” depends upon the degree of compaction and moisture contents anticipated in the field. This method gives a most complete picture of the behavior of the soils tested with changes in moisture and density, but it does involve an extensive testing program if the soils on the project are highly variable. For this reason many highway and other design agencies choose to use a single standard compactive effort and to mold all specimens at optimum moisture content for that effort. Nearly all agencies soak the compacted specimens under water for a period of four days before testing, to simulate the most adverse field conditions; exceptions are sometimes made in the cases of materials that are not affected by soaking or soils found in arid regions where adverse moisture conditions are not expected.

A soil survey (according to the OMR Geotechnical Bureau’s current guidelines) should be made to identify all soil variations to be found on the project, and CBR values should be determined for each soil significantly different from others. The CBR value, which is intended to be representative of the project’s subgrade strength, is usually selected from the lower quartile of the range of results. Once the engineer (usually a geotechnical engineer) selects the CBR value to be used for pavement design purposes, the value is converted to a Soil Support Value (SSV). Correlation charts, such as Figure 4.3, are used for this conversion while taking into account all collected data from the Soil Survey for the project. The SSV should be compared to the historic data presented in Appendix G before a final SSV recommendation is made to the pavement design engineer.

In limited instances, in the interest of economy, more than one CBR/SSV may be selected if distinct differences in soil type can be defined as representative of different portions of the same project (for instance, exceptionally weaker or stronger soils are encountered over large distances). However, is such variations occur over small areas of the project multiple design SSVs are not provided. Areas that include weaker soils are typically stabilized, for example, with thicker layers of base materials rather than complicating the design of the roadway by providing multiple design SSVs.

Numerous other test methods have been devised to measure soil strength. Among those which have a history of successful use in flexible pavement design procedures are the R-value (AASHTO Standard T 190 or ASTM D 2844) and the Texas Triaxial (AASHTO Standard T 212). Still other tests are designed to measure moduli of elasticity, or resiliency. The latest AASHTO Guide for Design of Pavement Structures (2002) recognizes and recommends the use of the Resilient Modulus (M_R) as a fundamental measure of subgrade strength.

While an accurate evaluation of soil strength is most important, there may be cases where tests for the strength of actual job-site soils are either impractical or not absolutely essential. On small, isolated jobs where strength testing facilities are not available, strength estimates on the basis of standard soil classifications may have to suffice. These classifications require only the determination of basic engineering properties such as grain size distribution, liquid limit, and plasticity index (AASHO Standards T 88, T89, and T 90, or ASTM D 422, D 423, and D 424). A correlation chart, Figure 4.2, is offered to indicate the approximate range in CBR/SSV or other strength tests values that may be applicable to soils that fall into various classifications. Obviously the correlation is very rough. Information on local soil types may be available from a number of sources. Many highway departments (including the GA DOT) maintain comprehensive records of engineering properties, including strength, of soils encountered on all projects. As referenced previously, Appendix G is a historic compilation of SSVs by County that GDOT has developed and should be referenced when developing subgrade strength parameters. Also, where the soils are not too variable, information from the files of one project may be adequate for others nearby.

Some agencies rely more on the soil area concept than on actual physical testing. The Soil Conservation Service of the U.S. Department of Agriculture has prepared pedological soil maps for numerous counties in many states, and where mapping has been done since the late 1950’s, each soil type has also been classified on the basis of tests for engineering properties. Information of this sort, where complete and accurate, may reduce or completely obviate the need for strength testing of job-site soils. However, in view of the supreme importance of soil support, it is emphasized that the designer should obtain the most accurate information on soil strength available.

The table below illustrates the various descriptions of subgrade classifications based on the CBR test. Soils are divided in three classes: good, fair, and poor.

CBR & SOIL CLASSIFICATIONS CORRELATIONS			
Class	**C.B.R.**	**Soil Support Value (1)**	**Description**
Good	10-plus	4.0 and 4.5	Retains a substantial amount of load bearing capacity when wet. Sands, sand gravels, materials free of detrimental amounts of plastic material. *P.I. less than 15*
Fair	6-9	3.0 and 3.5	Retains a moderate degree of firmness under adverse moisture conditions. Loams, silty sands, sand gravels with moderate amounts of clay and fine silt. *P.I. 15-20*
Poor	2-5	2.0 and 2.5	Soils containing appreciable amounts of clay and fine silt (50% or more passing –200) *P.I. Over 20*

(1) Typically, GA DOT recommends SSVs in the range of 2.0 to 4.5

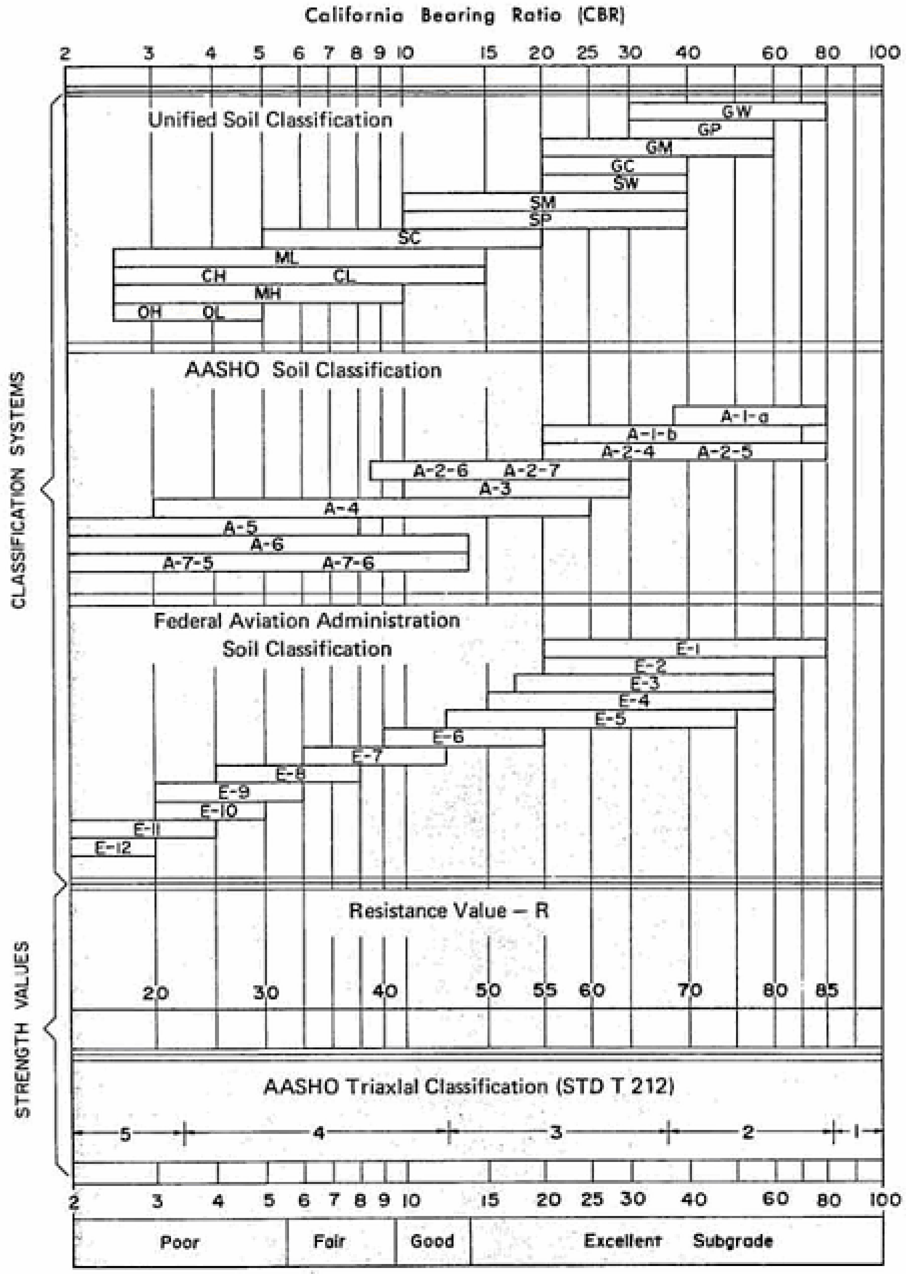
TABLE 4.1 CBR & SOIL CLASSIFICATIONS CORRELATIONS

The following are soil types and their compositions.

- **Good subgrade soils** retain a substantial amount of their load-supporting capacity when wet. Included are the clean sands, sand-gravels, and those free of detrimental amounts of plastic materials. Excellent subgrade soils are relatively unaffected by moisture or frost and contain less than 15 percent passing a No. 200 mesh sieve.
- **Fair subgrade soils** are those that retain a moderate degree of firmness under adverse moisture conditions. Included are such soils as loams, silty sands, and sand gravels containing moderate amounts of clays and fine silts. When this soil becomes a cohesive material, it should have a minimum proctor density of 110 pounds per square inch.
- **Poor subgrade soils** are those that become quite soft and plastic when wet. Included are those soils having appreciable amount of clay and fine silt (50 percent or more) passing a No. 200 sieve. The coarse silts and sandy loams may also exhibit poor bearing properties in areas where deep-frost penetration into the subgrade is encountered for any appreciable periods of time. This also is true where the water table rises close to the surface during certain periods of the year.
- **Very poor soils** (those with a CBR of less than 2) often perform poorly as pavement subgrades. However, to improve their performance, these soils can be stabilized with granular material. Lime, fly-ash, asphalt cement, Portland cement, and combinations of cement stabilizers also can be added to improve the subgrade support. The selection of a stabilizing agent, the amount to use, and the application procedure depends on the soil classification and the subgrade-support value desired. These should be determined through appropriate laboratory testing.

4.3 Parameters & Correlations

The following pages are correlation charts, tables, and data gathered relative to subgrade strengths, suitability, and other subgrade characteristics.



Complete evaluation of soil support requires more than a single standard measurement of strength. The tendency of soils of certain types to swell upon absorbing moisture may result in unevenness of the riding surface and in certain cases localized disruption of the entire pavement structure. It is recommended that soils that swell more than 3 percent during the CBR soaking period be classed as “poor.” Their use at or close to the subgrade level should be avoided wherever possible. Certain soils also suffer significant loss in support value after being frozen and thawed in the presence of moisture. Where this is likely to occur, the basic design thickness may have to be modified.

TABLE 4.2

CHARACTERISTICS PERTINENT TO USE IN PAVEMENT DESIGN

Major Division			GA DOT Classification	USCS Group Symbols	Typical Names	Performance when not subject to Frost Action			Potential Frost Action	Compressibility and Expansion	Drainage Characteristics	Recommended Compaction Equipment	Typical Unit Dry Weight [pcf]	Typical CBR	Typical SSV (GDOT Max Allowable SSV=4.5)	Subgrade Modulus Reaction [pci]
						As Subgrade	As Subbase	As Base								
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS Fifty percent or more of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	—	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Excellent	Excellent	Good	None to very slight	Almost None	Excellent	Vibratory roller, rubber-tired roller, steel-wheeled roller	125-140	40-80	7 - 8.5	300-500
			—	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	Good to Excellent	Good	Fair to Good	None to very slight	Almost None	Excellent	Vibratory roller, rubber-tired roller, steel-wheeled roller	110-140	30-60	6.5 - 8	300-500
		GRAVELS WITH FINES	—	GM	Silty gravels-gravel-sand-silt mixtures: Low LL & PI	Good to Excellent	Good	Fair to Good	Slight to Medium	Very Slight	Fair to Poor	Rubber-tired roller, vibratory roller; close control of moisture	125-145	40-60	7 - 8	300-500
					Higher LL & PI	Good	Fair	Poor to not Suitable	Slight to Medium	Slight	Poor to Practically Impervious	Rubber-tired roller, sheepsfoot roller	115-135	20-40	5.5 - 7	200-500
			—	GC	Clayey gravels, gravel-sand-clay mixtures	Good	Fair	Poor to not Suitable	Slight to Medium	Slight	Poor to Practically Impervious	Rubber-tired roller, sheepsfoot roller	130-145	20-40	5.5 - 7	200-500
	SANDS More than 50% of coarse fraction passes No. 4 sieve	CLEAN SANDS	IA1 to IA3	SW	Well-graded sands and gravelly sands, little or no fines	Good	Fair to Good	Poor	None to very Slight	Almost None	Excellent	Vibratory roller, rubber-tired roller	110-130	20-40	5.5 - 7	200-400
				SP	Poorly graded sands and gravelly sands, little or no fines	Fair to Good	Fair	Poor to not Suitable	None to very slight	Almost None	Excellent	Vibratory roller, rubber-tired roller	105-135	10-40	4 - 7	150-400
		SANDS WITH FINES	IIB1 to IIB4	SM	Silty sands, sand-silt mixtures: Low LL & PI	Fair to Good	Fair to Good	Poor	Slight to High	Very Slight	Fair to Poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	15-40	5 - 7	150-400
					Higher LL & PI	Fair	Poor to Fair	Not Suitable	Slight to High	Slight to Medium	Poor to Practically Impervious	Rubber-tired roller, sheepsfoot roller	100-120	10-20	4 - 5.5	100-300
				SC	Clayey sands, sand-clay mixtures	Poor to Fair	Poor	Not Suitable	Slight to High	Slight to medium	Poor to Practically Impervious	Rubber-tired roller, sheepsfoot roller	100-135	5-20	2.5 - 5.5	100-300
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less		IIIC1 to IIIC4	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	Poor to Fair	Not Suitable	Not Suitable	Medium to Very High	Slight to Medium	Fair to Poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	15 or less	1 - 5	100-200
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to Fair	Not Suitable	Not Suitable	Medium to High	Medium	Practically Impervious	Rubber-tired roller, sheepsfoot roller	90-130	15 or less	1 - 5	50-150
	SILTS AND CLAYS Liquid limit greater than 50%		IV	OL	Organic silts and organic silty clays of low plasticity	Poor	Not Suitable	Not Suitable	Medium to High	Medium to High	Poor	Rubber-tired roller, sheepsfoot roller	90-105	5 or less	1 - 2.5	50-100
			IIIC1 to IIIC4	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	Poor	Not Suitable	Not Suitable	Medium to Very High	High	Fair to Poor	Sheepsfoot roller, rubber-tired roller	80-105	10 or less	1 - 4	50-100
				CH	Inorganic clays of high plasticity, fat clays	Poor to Fair	Not Suitable	Not Suitable	Medium	High	Practically Impervious	Sheepsfoot roller, Rubber-tired roller	90-115	15 or less	1 - 5	50-150
			IV	OH	Organic clays of medium to high plasticity	Poor to very Poor	Not Suitable	Not Suitable	Medium	High	Practically Impervious	Sheepsfoot roller, rubber-tired roller	50-110	5 or less	1 - 2.5	25-100
Highly Organic Soils			IV	PT	Peat, muck and other highly organic soils	Not Suitable	Not Suitable	Not Suitable	Slight	Very High	Fair to Poor	Compaction not practical	—	—	—	—

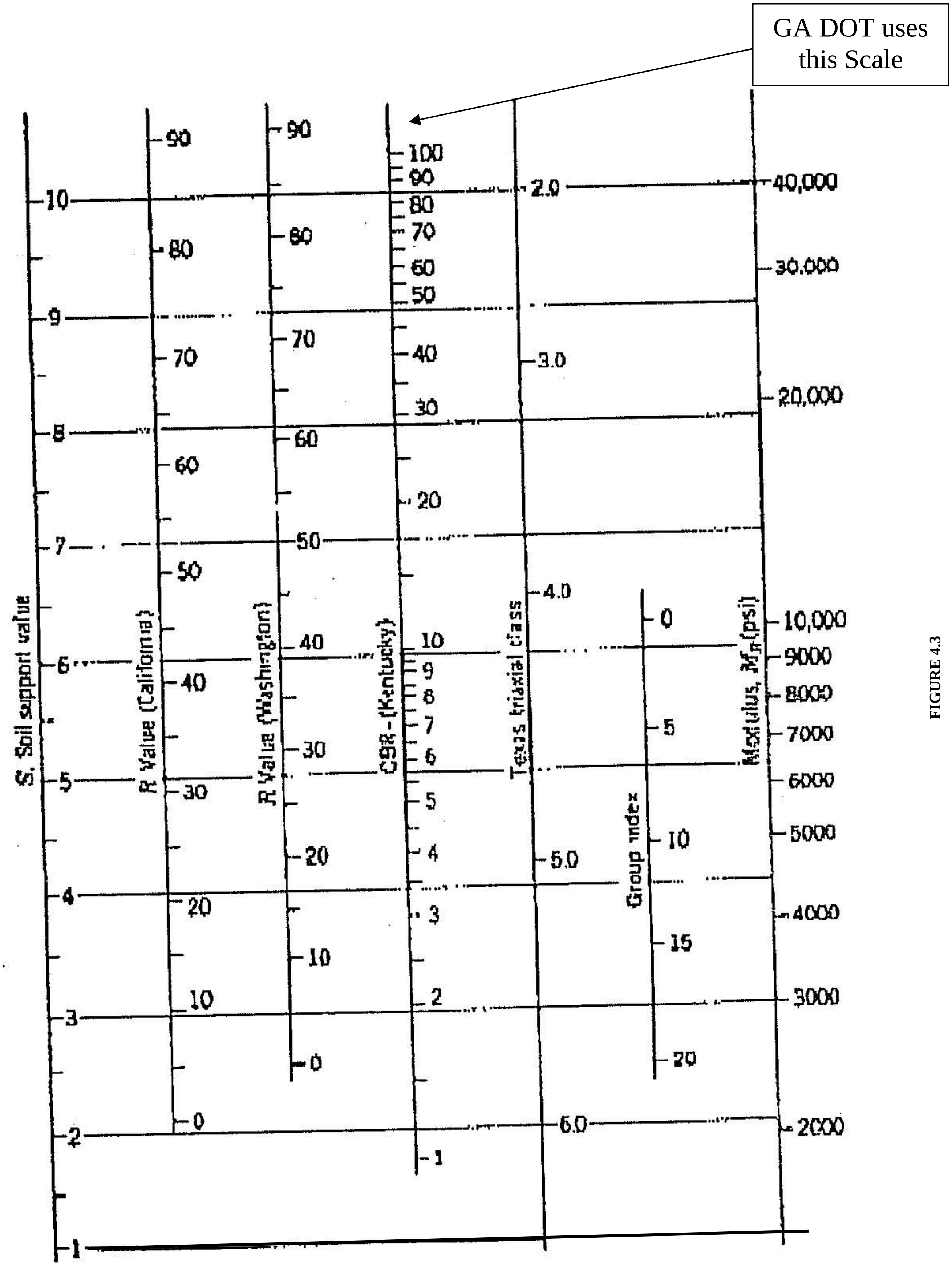


FIGURE 4.3

CORRELATION CHART FOR ESTIMATING SOIL SUPPORT VALUES (SSV)

4.4 Drainage

Drainage must also be considered for every pavement design. There are two basic categories of drainage: surface and subsurface. Surface drainage includes the disposal of all water present on the pavement surface, shoulder surface, and the adjacent ground when sloped toward the pavement. Subsurface drainage deals with water in the subbase, the surrounding soil, and in any of the pavement courses present. Inadequate attention to either of these two drainage conditions can lead to premature pavement failure.

4.5 Subgrade Stabilization

4.5.1 General

Subgrades can be stabilized mechanically (by adding granular materials), chemically (by adding chemical admixtures), or with a stabilization expedient (sand-grid, matting, or geosynthetics). Stabilization with chemical admixtures (lime, port-land cement, fly ash, and such) is generally costly but may prove to be economically feasible, depending on the availability of the chemical stabilization agent in comparison with the availability of granular material. The following sections summarize various aspects of subgrade stabilization; however, details regarding stabilization must be addressed on a project by project basis with coordination/consultation between the geotechnical and design engineers.

4.5.2 GDOT Subgrade Material Stabilizers

In general, soils with poor loading-bearing characteristics and/ or groundwater are reliable indicators of a potentially unstable construction environment. If either of these conditions is encountered during the subsurface field investigation (Soil Survey), then the use of stabilizer materials is an option for improving the subgrade's strength, as may be recommended in the Soil Survey Summary.

Section 209.2.B of the GDOT Standard Specifications for Highway Construction lists the following as stabilizer materials, which in turn are described under Section 803 – Stabilizer Aggregates or Section 810 – Roadway Materials:

- Type I Stabilizer Aggregate
- Type II Stabilizer Aggregate
- Type III Stabilizer Aggregate
- Type IV Stabilizer Sand
- Class IIB3 or better soils

Of these materials, Type III Stabilizer Aggregate is the most commonly used, followed by Class IIB3 or better soils. The other materials are rarely considered during the design process.

Type III Stabilizer Aggregate is more likely to be used to improve conditions on projects north of the Fall Line.

Class IIB3 or better soils are more likely to be used as stabilizing materials on projects located south of the Fall Line. This treatment may also be suitable for the Piedmont Region of the state if recommended in the Soil Survey Summary. It can be the most economical solution under the right conditions.

4.5.3 Granular Embankment

Granular embankment materials must meet the requirements of a Class IA2 soil, as per Section 810.2.01.A of the Standard Specifications with a couple of modifications that are listed in Section 212 – Granular Embankment.

This material is used as a replacement material (sometimes referred to as subbase) when unstable soils must be removed for subgrade stabilization. In counties below the Fall Line, it has been placed in layer thicknesses, approved by the Engineer, as much as 9 feet on occasions. However, in counties north of the Fall Line, granular embankment is typically placed in layers thicknesses less than 4 feet because of the availability of rock embankment in situations that call for larger quantities of removal. There are other factors that can determine the maximum layer thickness north of the Fall Line, such as the quantities of material needed, ease of construction, ability to achieve compaction of the embankment materials, which can only be determined on a project-by-project basis.

In counties south of the Fall Line, granular embankment, usually in conjunction with a geotextile, is placed where the embankment is constructed in inundated areas, which are not drainable prior to construction. It is typically placed to a height of 18 inches above the expected high water level. In counties north of the Fall Line, the required layer thickness of replacement material, for instance, can determine if granular embankment will be used instead of rock embankment.

4.5.4 Rock Embankment

Rock embankment materials are also used to provide subgrade stabilization. Rock Embankment materials are unweathered quarry-run stone that are smaller than 4 feet in any direction, as required by Section 811 – Rock Embankment of the Standard Specifications.

It is typically used in counties north of the Fall Line because of its readily available. This material can be used as a replacement material (subbase) where removing unstable soil layers thicker than 4 feet.

It is also used for placing the road embankment in inundated areas, which are not drainable prior to construction and is usually placed over a geotextile when placed over loose material. It is typically placed to a height of 18 inches above the expected high water level.

Caution must be taken when placing Rock Embankment materials, as the larger, open-graded stone must be “choked” with smaller materials at the top to minimize migration of soil particles in the event of inundation. The placement of a geotextile over it stone can also serve as an effective barrier to prevent the migration of soil particles into the rock embankment materials.

4.5.5 Select Materials Subgrade

The use of select materials subgrade is a common practice for projects in counties south of the Fall Line. These materials consist of medium- to well-graded soils that are readily found in this part of the state, as opposed to the counties in the Piedmont Region, which typically do not have soils with load-bearing properties as good as those soils in the Coastal Plain Region.

When a blanket of select materials subgrade is specified, Special Provision 209 – Subgrade Construction is provided to require that areas of the project that do not already have suitable at-grade soils receive a 12-inch blanket of Class IIB2/ IIB3 or better soils. These at-grade soils are not necessarily of poor load-bearing quality, but rather they are not expected to provide the desired Soil Support Value (SSV). For example, if a project required a SSV of 3.0 or 3.5, then the geotechnical engineer would specify that a 12-inch blanket of Class IIB3 soils or better would be required; or if a SSV of 4.0 or 4.5 was required, then the engineer would specify that a 12-inch blanket of Class IIB2 soils or better would be required.

4.5.6 Lime Stabilized Subgrades

Lime is often an excellent choice for improving unfavorable roadbed materials to form a base, subbase, or subgrade. Lime stabilization is the modification of inherently weak or excessively plastic soils that may also be wet, into a much-improved material whose engineering properties are significantly altered. Lime reacts with clay minerals resulting in increased strength and a reduction of soil plasticity, moisture content, and volume change with moisture variation.

Lime will have a positive effect on a broad range of soils, but is most effective with clay soils, with which it can react both chemically and physically to produce a fundamentally new material. Provided sufficient clay is present, the remainder of the soils can be gravels, sands, or silts. Organic contamination and/or highly acidic soils should be avoided

Lime is created from limestone (calcium carbonate) that is burned at extremely high temperature and crushed into a fine powder. The result is quicklime (calcium oxide) or hydrated lime (calcium hydroxide) that is formed by slaking quicklime with a controlled amount of water. Both quicklime and hydrated (slaked) lime are suitable for stabilization purposes. It should be noted that limestone rock that has been quarried and pulverized for agricultural use is often referred to as lime but has no beneficial effect in stabilizing soils, although it will result in some drying of wet soils, by absorbing moisture.

Application of lime is fairly simple. The roadbed soils to be treated should be pulverized into particles generally about the size of a walnut and thoroughly mixed with the lime. Water is added and the treated materials is then compacted and allowed to mellow (typically 24 hours) in order for the lime to react. Subsequently, the base, subbase, subgrade course must be remixed to ensure that the lime has been thoroughly incorporated, leaving no clumps of non-reacted lime which can result in isolated areas of heaving. Then the final compaction of the newly altered material can be performed.

While any amount of lime may provide some benefit, mixing 4% to 8% lime (by weight) with subgrade soils typically provides the desired result. Percentages greater than about 8% generally do not result in significant additional improvement.

Dust control and inhalation protection are required when lime is applied to the soil, Quicklime, in particular, because of its higher reactivity, is more difficult to control and its use may be problematic in populated areas. Rapid hydration may produce high heat, causing the soils to sputter and boil, emitting steam that can cause chemical burns.

Construction procedures, testing methods and specification regarding the use of lime for soil stabilization are outlined in Sections 225 and 822.2.02 of the GDOT Standard Specifications, 2001 Edition

4.5.7 Fly Ash Stabilized Subgrades

Etymology

The Romans knew that volcanic ash (pozzolans), when finely ground and mixed with lime and sand yielded a mortar that was cementitious, water resistant, and strong. Almost 1000 years ago the Mayans utilized pozzalanic materials as mortar to construct their temples in Central America.

Fly Ash Facts

References found in the GDOT Standard Specifications are Section 319, 326 and 831.2.03.

Fly ash stabilized base course is suitable for both flexible and rigid pavements. Pozzalanic-stabilized mixtures (PSM) can use several materials and material combinations to construct stabilized aggregate bases. Class C (AASHTO M 295) fly ash can be used as a stand-alone material. Class F fly ash can be used when blended with lime, Portland cement or cement kiln dust (CKD). The stabilization of aggregate bases provides several advantages:

- Adds significant strength and durability
- Allows the use of marginal or low quality aggregates
- Permits better use of open graded base courses

Closely controlled curing conditions are important as both time and temperature significantly affect strength and durability. Also, a high degree of compaction is crucial to performance of PSM. Final density should be achieved as quickly as possible to achieve the highest ultimate strengths. With Class C fly ash this is especially true because it is rapid setting. Curing- a prime coat should be applied quickly to seal the surface to prevent drying.

Fly ash that contains sulfur in excess of 5 % as SO_3 or contain scrubber residues should be carefully evaluated with project specific soils to assess the expansion potential of the materials combination.

Typical proportions for Class F fly ash-lime blends are 2 to 8 % lime blended with 10 to 15 % Class F fly ash. Also, Class F fly ash can be blended with 0.5 to 1.5 % Type I Portland cement to produce a stabilizing agent.

Reactivity and fineness are the major fly ash characteristics that most directly affect PSM quality.

4.5.8 Stabilization Utilizing Man-Made Stabilizers

A stabilization expedient may provide significant time and cost savings as a substitute to other means of stabilization or low strength fill. The most popular of the man-made stabilizers are sand grid, roll-matting, and various types of geosynthetics, especially geotextiles. Matting and sand grid are expedient methods of stabilizing cohesionless soils such as sand for unsurfaced road construction. Geotextiles and other geosynthetics are primarily used to reinforce weak subgrades, maintain the separation of soil layers, and control drainage through the road or airfield design. The availability of these materials must be weighed with the considerable time savings for use of expedients in combat construction. The *Geosynthetic Design and Construction Handbook* (publication No. FHWA HI-95-038, revised April 1998) is an excellent resource when determining the type and use of geosynthetics in subgrade stabilization.

GDOT Specification Section 809-Grid Materials addresses reinforced slopes and Mechanically Stabilized Embankment (MSE) Wall backfill. Even though GDOT does not use this procedure specifically, AASHTO M288-96 addresses geotextiles utilized as material for separation of soil subgrades, stabilization of soft subgrades, and prevention of reflective cracking. Separation of soil subgrades is accomplished by placing a flexible porous geotextile between dissimilar layers so that the integrity and functioning of both layers can remain intact. Stabilization (reinforcement) is accomplished by the improvement of a system’s total strength created by the introduction of a geotextile (good in tension) into a soil (good in compression but poor in tension) or into other disjointed and separated materials. The filtration function of a geotextile involves the movement of liquid through the fabric.

In M288-96, the classifications are essentially a list of strength properties meant to withstand varying degrees of installation survivability stresses.

- **Class 1** - for severe or harsh survivability conditions where there is a greater potential for geotextile damage.
- **Class 2** - for typical survivability conditions; this is the default classification to be used in the absence of site-specific information.
- **Class 3** - for mild survivability conditions.

Class 1 geotextiles are utilized for stabilization of subgrades. Class 2 geotextiles are for separating soil subgrades. Class 3 geotextiles are recommended for prevention of reflective cracking unless harsh survivability conditions are anticipated.

Hydraulic Conductivity (permeability, subsurface filtration or drainage) must be considered in problematic soil environments, i.e., fine-grained soils, silts and clays. The soil-to-geotextile system must allow for adequate fluid flow with limited soil loss across the plane of the geotextiles over its service life.

Minimum fabric properties, woven or nonwoven, should be based on Minimum Average Roll Values (MARV) and not average lot values. Average lot values are considerably higher than the minimum value. An intermediate value between these two extremes is the MARV. This value is probably two standard deviations lower than the average lot value.

4.6 Frost Susceptibility of Subgrade

In areas subjected to seasonal freezing and thawing, subgrade materials may exhibit frost heave and thaw weakening. However, such is not a problem in Georgia and is therefore not a significant consideration in roadway design for the GA DOT.

4.7 Geotechnical Testing Requirements

As referenced earlier in this section of the manual, a Soil Survey is required for all projects to determine the soil types and subsurface conditions along the project alignment. The soil survey should be completed in accordance with the Geotechnical Engineering Bureau's QA/QC Manual (Guidelines for Geotechnical Studies), which is available on line at:

http://www.dot.state.ga.us/dot/construction/materials-research/b-geotech/qaqcmanual/00qaqctoc.shtml

The Geotechnical Engineering Bureau, which is part of the GA DOT Office of Materials & Research (OMR), should be contacted for guidance regarding scope of work undertaken for each project and they should provide final report review for quality assurance purposes.

5 Bases

5.1 Graded Aggregate Base

Graded Aggregate Base course (GAB) plays a very important role in the overall integrity of the concrete slab or bituminous pavement layer. The GAB is placed between the prepared subgrade and the top pavement layer. Base course may also include several types of under courses, such as subbase and filter beds. Base course serves a variety of purposes depending on the construction practices and the environment. These consist of providing structural capacity to bituminous asphalt slab, drainage for Portland cement concrete slab, and low susceptibility to frost. It is noted that gradation of the GAB is the very important factor in success of aggregate as a base course. Since the gradation of the aggregate can affect structural capacity, drainage, and frost susceptibility, control of gradation is a principal concern for most engineers.

Three types of gradations could occur. First, aggregate with no fines; second, fines just filling the voids of the aggregate fraction; third, fines overfilling the voids of the aggregate fraction. However, in GDOT construction, graded aggregate base having a specified gradation and consisting of particles ranging in size from 37.5 mm to 75 μm, is the accepted material. One gradations are specified for silicate aggregates, granite, granitic gneiss, quartzite, and so on, also called Group II aggregates); two gradations are specified for carbonate aggregates, (limestones, dolostones, and marbles, also called Group I aggregates). Two possible gradations are specified for Group I base due to the tendency of some carbonate rocks to produce inadequate amounts of fines when crushed.

Crushed recycled concrete is also acceptable graded aggregate base, provided it meets the gradation, sulfate soundness, and Los Angles (L.A.) abrasion requirement.

During construction, compaction to 100 % of the theoretical maximum dry density must be achieved for Group II aggregates, and recycled concrete base. Applicable material specifications are given in Section 805 and construction procedures are given in Section 310 of the GDOT Standard Specifications, 2001 edition.

In the first case the aggregate would derive its strength from the interlocking of the aggregate particles. Therefore, for the base course to be stable, the base course material should be confined. However, this type of aggregate gradation would provide excellent drainage and is completely non-frost-susceptible.

In the second case the aggregate would still derive its strength from interlocking of the aggregate particles. However, due to cohesiveness of the fine particles, the structural integrity of aggregate would not be compromised if unconfined. In addition, the drainage is adequate and can be non-frost-susceptible.

In the third case the strength of the aggregate is primarily derived from the interlocking effect of the fine particles rather than the larger particles, therefore, a strength reduction occurs. The drainage characteristic of these types of aggregate would be poor and would therefore be very frost susceptible.

Finally, for aggregate to resist stresses induced by repeated loads and to avoid aggregate degradation, base course aggregate must exhibit strength and toughness for their intended use. The Georgia Department of Transportation has set limits that the aggregate must meet for use as GAB.

GDOT does not recognize Crusher Run as an acceptable base material because it does not consistently meet its gradation requirements. Crusher Run maximum aggregate size is usually in the 2-inch to 3-inch range. Some local municipalities or county governments may utilize Type III Stabilizer Aggregate (Section 803.2.03 of the Standard Specifications) or ungraded Crusher Run for base material. Therefore, gradation tests on aggregates for non-state controlled roads should be undertaken to determine the acceptability of base materials.

5.2 Soil Cement

Soil-cement, a mixture of a measured amount of cement, pulverized soil materials and water compacted to high density is often used as a subbase course for road construction. The mixture gradually becomes a hard structural material as the cement hydrates with time. Once cured, it reduces rutting or shoving during spring-thaw cycle. The improvement is a function of the quantity of cement added and the roadbed material type, such as sand, silt, clay, gravel, crushed stone, and so on.

Because soil-cement is a structural material, it must possess a few engineering properties dependent on the soil material, compaction, cement content, age, curing conditions, and so on. Typical cement content may range between 5 and 9 percent by weight of soil, depending on the amount of silt and/or clay present. Generally, the cement content becomes higher with more cohesive soils. Typical soil-cement layer thicknesses range between 6 and 8 inches.

Typical 28-day compressive strength of saturated soil-cement specimen ranges from 300 psi (2070 KPa) to 900 psi (6205KPa) with modulus of elasticity (E) in the 0.6 to 2 million psi (4,200 to 14,000 MPa). The modulus of rupture (M_R) is usually about 20% of compressive strength. The laboratory unconfined compressive strength for an approved mix design of 450 psi (3103 KPa) should yield field strengths of 300 psi (2070 KPa) for quality acceptance. As the cement continues to hydrate over time, soil-cement continues to gain strength higher than the 28-day period. This makes soil-cement an excellent choice over other base materials particularly where the increase in volume and weight of traffic is anticipated.

Construction procedures, testing methods and specification regarding the use of cement for soil stabilization are outlined in Sections 301 and 814.2.02 in the GDOT Standard Specifications, 2001 Edition.

The designer should realize that soil cement shrinks as it cures and forms irregular crack patterns. These cracks can be reflected through asphalt pavement if not properly taken into account. Generally, the minimum acceptable flexible pavement thickness over soil cement base is 6 inches.

5.3 Guidelines for Soil Cement Alternate

The use of soil-cement base as an alternate base course material requires that two conditions be satisfied. First, the project must be located in an area containing suitable soils for soil-cement base. Second, project construction must be conducive to the use of soil-cement such that constructability and safety issues are addressed. The Design Engineer should use the following guidelines when making the decision on when and where to use soil-cement:

5.3.1 Materials

The areas below the solid line on the attached map (Appendix I), exclusive of the cross-hatched areas, generally contain suitable soils for soil-cement base.

Note: The soil report may contain a recommendation regarding the use of soil cement as an alternative base material.

If the proposed project is located in a county that is split by the solid line, the Design Engineer should contact the Pit and Quarry Branch of the Office of Materials and Research (404) 363-7590 to verify the existence of suitable soils for soil-cement in the vicinity of the project prior to setting up soil-cement as an alternate base.

If the proposed project is located in an area below the solid line, exclusive of the cross-hatched area, the materials are generally suitable and no further checking for materials compatibility for soil-cement is required prior to setting it up as an alternate based.

The cross-hatched areas do not contain suitable soils for soil cement base and therefore, soil cement base shall not be considered as an alternate in these areas.

5.3.2 Constructability

The Design Engineer should consider soil-cement base as an alternate only for the following types of construction.

New location work: This is the addition of new lanes in a rural setting with a split median, such as the GRIP, EDS or other work where two new lanes are being added adjacent to the existing lanes.

Widening projects of any kind that require base to be placed adjacent to existing pavements shall not be considered as candidates for a soil-cement base alternate. Tie-ins for construction under are excluded from this restriction.

5.3.3 Design Considerations

When setting up soil-cement base as an alternate, do the following:

- Set up soil-cement based by the Square Yard or Square Meter
- Set up Portland cement by the ton or megagram
- Obtain a recommended percent cement and a dry unit weight of soil from the Office of Materials and Research (404) 363-7590 for use in calculating the quantity or Portland cement for the project.

The thickness of soil-cement base should not be less than 6 inches (150 mm) or more than 8 inches (200 mm).

When mainline and ramps are involved, use the same thickness of soil-cement base for each condition.

Do not set up soil-cement base on shoulders under any condition. Use full depth asphalt for this application. The minimum equivalent typical section should be 1.5 inches (40 mm) of 12.5 mm asphaltic concrete, 2 inches (50 mm) of 19 mm asphaltic concrete, and 3 inches (75 mm) of 25 mm asphaltic concrete for shoulder construction whenever soil-cement base is used on the mainline or ramps.

Use the thickness equivalents listed below when setting up alternate bases that include soil-cement. It is important that these be used to ensure equivalent base structures and competitive bidding on the alternates.

Table of Equivalent Thickness

Asphaltic Concrete		Soil- Cement		Graded Aggregate
4 inches (100 mm)	=	6 inches (150 mm)	=	8 inches (200 mm)
5 inches (125 mm)	=	8 inches (200 mm)	=	10 inches (250 mm)

5.3.4 Enforcement

The use of soil-cement base should be addressed at the Preliminary Field Plan Review stage. The State Construction Engineer should approve any deviation from these guidelines.

	Super Pave	Layer Thickness	Material SN	Layer SN
Asphaltic Concrete	12.5mm	1.5”*	0.44	0.66
Asphaltic Concrete	19 mm	2.0”	0.44	0.88
Asphaltic Concrete	25 mm	1.0”	0.44	0.44
Asphaltic Concrete	25 mm	2.0”	0.30***	0.60
Soil Cement Base		6.0”	0.20	1.20
	**	Proposed	SN	3.78

Asphaltic Concrete	12.5mm	1.5”*	0.44	0.66
Asphaltic Concrete	19 mm	2.0”	0.44	0.88
Asphaltic Concrete	25 mm	1.0”	0.44	0.44
Asphaltic Concrete	25 mm	2.0”	0.30***	0.60
Soil Cement Base		8.0”	0.20	1.60
		Proposed	SN	4.18

Asphaltic Concrete	12.5mm	1.5”*	0.44	0.66
Asphaltic Concrete	19 mm	2.0”	0.44	0.88
Asphaltic Concrete	25 mm	1.0”	0.44	0.44
Asphaltic Concrete	25 mm	3.0”	0.30***	0.90
Soil Cement Base		8.0”	0.20	1.60
		Proposed	SN	4.48

Asphaltic Concrete	12.5mm	1.5”*	0.44	0.66
Asphaltic Concrete	19 mm	2.0”	0.44	0.88
Asphaltic Concrete	25 mm	1.0”	0.44	0.44
Asphaltic Concrete	25 mm	4.0”	0.30***	1.2
Soil Cement Base		8.0”	0.20	1.6
		Proposed	SN	4.78

Asphaltic Concrete	12.5mm	1.5”*	0.44	0.66
Asphaltic Concrete	19 mm	2.0”	0.44	0.88
Asphaltic Concrete	25 mm	1.0”	0.44	0.44
Asphaltic Concrete	25 mm	6.0”	0.30***	1.80
Soil Cement Base		8.0”	0.20	1.60
		Proposed	SN	5.38

TABLE 5.2 SAMPLE SOIL CEMENT BASE ALTERNATES

* The surface course may change to a different thickness according the selected material.

**Minimum section used on State Routes

*** 0.44 is used for the top 4.5 inches of asphalt.

The minimum allowable asphaltic concrete over soil cement bases is 6 inches

Additional Asphaltic Concrete Base can be added in 1-inch increments to attain the desired structural number.

5.4 Asphalt Base

The function of an asphalt base is to provide a stress-distributing medium that will spread the applied surface load so that shear and consolidation deformations will be minimized in the subgrade. It is generally considered the most cost-effective and dependable type of base course for heavy loads and high traffic volumes in some South Georgia counties where Soil Support Values may be greater than or equal to 3.5.

Usually the base course is the same as the binder course in conventional pavement. The minimum lift thickness is 3 inches to a maximum lift thickness of 5 inches. See GDOT Specification Section 400.3.05, Table 5 for specific allowable layer thickness. The final design thickness is based primarily on structural, construction, and maintenance considerations. Although not commonly used, a sand-bituminous stabilized base is also allowed by GDOT specifications under Section 302. The maximum allowable lift thickness of this mix is a compacted eight (8) inches. Multiple layers of this mix are allowable.

For rigid (PCC) pavements, subject to large numbers of heavy wheel loads and high traffic volumes, asphalt base provides several advantages. It controls movement of water upward toward the surface, prevents soil movement through joints in PCC, drains water that has entered PCC through joints and cracks.

The use of asphalt bases requires Special Provision 400 – Hot Mix Asphaltic Concrete Construction and Special Provision 828 – Hot Mix Asphaltic Concrete Mixtures. These Special Provisions close the air voids of the Superpave mix to prevent the flow of water through Superpave mixes that can saturate the subgrade and lower the design strength of the soil.

Note: See Chapter 13.4 and figures 13.8 to 13.10 for alternate base design examples.

5.5 Macadam Base

5.5.1 Etymology

John MacAdam invented the original macadam pavement in the 19th century utilizing layers of various size stones. A layer of large stones, 6 inches to 8 inches was placed and then layers of smaller stones and finally a layer of “dust” to choke the interstices. The “dust” was held down by water. Frequent sprinkling of the road was required.

5.5.2 Empirical

Today in Georgia some counties and towns may utilize greatly simplified macadam (surface treatment). There is no standard for macadam in the GDOT specifications. The typical approach is to prepare the subgrade by blading and compacting the surface, applyng a prime coat and then an aggregate-wearing surface. The typical prime coat (MC30, RC30, MC70, and RC70) penetrates the subgrade, plugs the voids, and provides a tacky surface for binding the surface treatment aggregate, typically #89 or #9 stone. After application of the aggregate the surface is rolled to bond the elements. Thickness of the prime coat determines the amount of aggregate that will remain in place. The thickness of macadam could be ½ inch to 1 inch.

Recognizing macadam is quite easy because bitumen is usually exposed in wheel lanes and aggregate appears between wheel lanes. Quite often loose aggregate can be spotted on the shoulders several years after a macadam surface has been constructed.

5.6 Limerock Base

Limerock is sedimentary rock mined from coastal deposits consisting primarily of carbonates of magnesium and/or calcium. In Georgia, the use of limerock for roadway construction is generally limited to the southern and southeastern coastal regions where ample sources of limerock are available (typically hauled in from Florida). Limerock, like other aggregate materials, may be useful for stabilization of unstable roadway subgrade soils or may be used to construct roadway base courses. While CBR testing is typical for most aggregate materials, the bearing value for limerock material requires a slightly different testing method, specifically the Limerock Bearing Ratio (LBR) Test, Florida Method FM 5-515. Limerock used in roadway construction must consist of at least 80% carbonate content (magnesium and/or calcium), should be relatively free of sand, clay, and organic material and must have a minimum LBR value of 100.

Note: Limerock base brought in from Florida shall be measured on a thickness per square yard basis not per ton.

Construction procedures, testing methods and specifications regarding the use of limerock for roadway construction are outlined in Section 815.2.02 of The GDOT Standard Specifications, 2001 Edition.

5.7 Asphalt Emulsion

Emulsified Asphalt (also simply called emulsion) is a mixture of asphalt cement, water and an emulsifying agent. Emulsion is made by combining these materials and passing them through a high shear colloid mill. This produces extremely small (5 to 10 micron) droplets of asphalt which are suspended by imparting an electrical charge to the surface of the droplets causing them to repel one another.

Emulsions are liquid at ambient temperatures. They can be applied at cooler temperatures than asphalt cements and cutback asphalts.

When an emulsion comes in contact with aggregate, the asphalt droplets react with the aggregate surface squeezing out the water between the aggregate particles. “Breaking” or “setting” also occurs due to evaporation of water from the emulsion.

See Section 302 of the GDOT Standard Specification for Sand-Bituminous Stabilized Base Course using emulsified asphalt.

There are two major categories of emulsion: Cationic and Anionic. Anionic emulsions have negatively charged asphalt droplets and are specified in AASHTO M 208-86. Cationic emulsions have positively charged asphalt droplets and are specified in AASHTO M 140-86. Section 824 of the GDOT Standard Specification addresses cationic emulsions.

Both Anionic and Cationic Emulsions are further graded according to their “setting” rate: rapid setting, medium setting, and slow setting. Setting rates are controlled by the type and amount of emulsifying agent.

Emulsified asphalts can be used with cold as well as heated aggregate and with aggregate that is either dry or damp. Its damp aggregate capability gives emulsified asphalt an advantage over cutback asphalts.

Emulsified asphalts are used for both road construction and specialty applications. The rapid setting grades are used in spray applications such as aggregate chip seals, sand seals, and similar surface treatments.

5.8 Full Depth Reclamation

Full Depth Reclamation (FDR) is a pavement rehabilitation technique, in which the full flexible pavement section and a predetermined portion of the underlying materials are uniformly crushed, pulverized or blended, resulting in a stabilized base course. FDR is utilized to rebuild a pavement that has reached the end of its useful life by recycling the materials in the existing roadway, and adding stabilizing additives such as mixing with cement and water or bituminous mixtures, and compacting to produce a strong, durable base. With today’s (2005) equipment FDR is limited to about 12 to 14 inches.

Typical bituminous mixtures are emulsified asphalt and cutback asphalt. The recycled base would be strong, uniform and more moisture resistant than the original material. Additional aggregate can be incorporated into the recycled material to improve its base characteristics when needed.

Full Depth Reclamation is distinguished from other reclamation techniques such as Cold Planing, Cold In-place Recycling, or Hot In-place Recycling by the fact that in FDR the cutting heads penetrate all the way through the asphalt section and into the underlying base layers. This technique erases deep pavement crack patterns and eliminates potential reflection cracking.

Full-depth reclamation uses the old roadway pavement and base materials as base material for the new pavement section. A new surface material could be a thin bituminous chip seal, HMA, or concrete.

Note: GDOT has a Committee working on Full Depth Reclamation. Their work would be incorporated in this manual when it’s available.

5.9 Cutback Asphalts

Cutbacks are petroleum solvents used to dissolve asphalt cement. The solvents are also referred to as distillates, diluents and cutter stock. The primary purpose of cutback asphalt is to provide tack between the old pavement (or base) and the new pavement being laid down, in order to prevent slippage of the layers. In addition, this coat of cutback asphalt serves as a moisture barrier.

Cutback asphalt is also used in stabilized base course applications. See Section 302 of GDOT's Standard Specification for Sand-Bituminous Stabilized Base Course using cutback asphalt.

If the solvent used to make the cutback asphalt is highly volatile, it will evaporate quickly. The less volatile the solvent, the slower the evaporation time. See Section 821 of GDOT's Standard Specification. Cutback asphalt is divided into three types:

- **Rapid curing (RC)** - Asphalt cement and a light diluent of high volatility-generally in the gasoline or naphtha boiling point range (RC-30,70, 250, 800, 3000).
- **Medium curing (MC)** - Asphalt cement and a light diluent of intermediate volatility-generally in the kerosene boiling point range (MC-30, 70, 250, 800, 3000).
- **Slow curing (SC)** - Slow curing asphalts are often called road oils-from the days when asphalt residual oil was used to give roads a low-cost, all-weather surface.

The degree of fluidity in each depends principally on the proportion of solvent to asphalt cement. The degree of fluidity results in several grades of cutback asphalt. Some are fluid at ordinary temperatures while others are more viscous, requiring some heating to make them fluid enough for construction purposes.

Note: See Section 823 of GDOT's Standard Specification.

Rapid-Curing Cutback Asphalt is used primarily for surface treatments and tack coat. Polymer Modified Rapid-Curing Cutback Asphalt is typically used for surface treatments.

Rapid-Curing Cutbacks are specified under AASHTO M81-75. Polymer Modified Rapid-Curing Cutbacks currently do not have a specification.

Medium-Curing Cutback Asphalt is typically used for prime coat, surface treatment, and stockpile patching mixes.

Medium-Curing Cutbacks are specified under AASHTO designation M82-75. Polymer Modified Medium-Curing Cutbacks currently do not have a specification.

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6 Pavement Types and Layers

6.1 Overview

Pavements are divided into two broad categories; flexible pavements and rigid pavements. Both pavement types are made up of the following layers:

- **Subgrade Layer** is the layer of native or stabilized roadbed soil. This layer is prepared and compacted to support a proposed pavement structure.
- **Subbase Layer** is the layer in the pavement system that is between the subgrade layer and a base course, or alternately between the subgrade and a PCC pavement.
- **Base Course Layer or Base Layer** is a layer of select material, such as graded aggregate, of planned thickness constructed on the sub-grade or sub-base below a pavement. It can serve:
  - As a construction platform
  - Distribute loads more evenly, and to a lesser extent
  - Assist in drainage
- **A Bond Breaker Layer** is used to prevent the adhesion of newly placed concrete from the underlying base material or other substrate in Portland Cement Concrete Pavement construction**.** GDOT uses 3 inches of Asphalt Binder Mix for this layer**.**
- **The Pavement System** consists of all natural, modified, and manufactured layers that constitute a pavement.

NOTE TO MANUAL USERS:

The definition of the "subbase" and "subgrade" differ somewhat within the industry. This manual will use the following distinctions for design purposes:

If it is a subgrade, the material strength properties will be defined with:
-A "Soil Support Value" for Flexible Pavements
-A "Modulus of Subgrade Reaction" for Rigid Pavements

If it is a base or subbase, the material strength properties will be defined with:
-a "Structural Number"

6.2 Flexible Pavements

Flexible pavements are so named because they flex under the actions of traffic and rebound when traffic loads are removed. They consist of a base material that has been overlaid by asphalt concrete layers.

Flexible pavements can be further grouped as follows:

- Surface Treatments
- Chip Seals
- Micro Surface Treatments or Micro Seals
- Thin Asphalt Concrete Pavements, and
- Asphalt Concrete Pavements

6.2.1 Surface Treatments

Surface Treatments are applications of an asphalt coating to the surface of an existing pavement so as to:

- Protect the surface characteristics
- Restore the functionality
- Retard the deterioration of an existing asphalt concrete surface

Load carrying is primarily accomplished by the existing pavement. The asphalt coating could be an asphalt tack coat, an emulsified asphalt or fog seal. If an aggregate is included in the application, then it may be referred to as a Chip Seal or a Micro Surface treatment.

6.2.2 Chip Seals

Chip Seals are essentially a single layer of asphalt concrete binder that is covered with a single size aggregate. The asphalt seals the underlying pavement surface and provides moisture protection. The aggregate provides the texture for tire-surface contact. This type of application is suited for low volume state routes or as a crack retarding treatment. The load carrying is primarily done by the sub-grade or base material or the existing pavement if the chip seal is used a crack retardant.

6.2.3 Micro Surfacing Treatments

Micro Surfacing Treatments consist of a mixture of a mineral aggregate, a polymer modified asphalt emulsion, mineral filler, and other modifiers. They are properly proportioned and thoroughly mixed and spread on an existing pavement. Micro Surfacing treatments are used on higher volume routes, are used to restore the pavement surface profile and seal cracks in the existing pavement surface. They generally provide a seven-year service life and are suited as a pavement preservation measure.

6.2.4 Thin Asphalt Concrete Pavements

Thin Asphalt Concrete Pavements ,like chip seals and surface treatments, are a surface layer of hot mix asphalt concrete, not exceeding 1.5 inches, placed directly over a sub-grade or base material. The load carrying is primarily done by the sub-grade or base material. This type of pavement is suited for low volume state routes.

6.2.5 Thin Asphalt Concrete Pavement Overlays

Thin Asphalt Concrete Pavement Overlays are generally a single layer of hot mix asphalt concrete that is used to add structure to an existing pavement, or to restore surface characteristics as a maintenance rehabilitation measure.

6.2.6 Asphalt Concrete Pavements

Asphalt Concrete Pavements consist of several asphalt concrete layers that are placed over the base material and sub-grade to provide a structural system. They are designed to carry a higher level of traffic and are suited for most state routes.

6.2.7 Flexible Pavement Structure

The combined thickness of all layers in a pavement structure is determined from:

- The number of traffic loadings the pavement will have to carry during its service life
- The geotechnical (soil support) conditions, and
- To a lesser extent on its location in the state (regional factor)

All asphalt concrete pavements used by GDOT are designed using the AASHTO Interim Guide for the Design of Pavement Structures 1972 Chapter III Revised 1981.

In general, the asphalt concrete layers, typically used on state routes, are the following:

- The Riding Surface layer.
- The Asphalt Binder layer which is immediately below the surface course
- The Asphalt Base layer that is immediately below the binder and surface courses

For higher volume higher duty state routes, the riding surface specified, may differ from typical state routes, and is dictated by:

- Volume and function such as the Stone Matrix Asphalt Layer (SMA)
- Other safety related consideration such as drainage, for which an Open Graded Friction Course (OGFC) or a Porous European Mix (PEM) is used.

For Interstate routes, GDOT uses the following asphalt concrete layers:

- An Open Graded layer such as OGFC or PEM for the Riding Surface. Those are high void plant mixes that allow drainage of rainwater. The rolling tire pressure pushes water into the voids to provide a "dry" footprint during wet weather. In terms of its gradation this is a coarse one-sized aggregate mix.
- An SMA layer for channeling water to the edge for draining, as well as to provide added structure
- The Asphalt Binder layer
- The Asphalt Base layer

The surface layer, whether OGFC or PEM, assists in draining water from the surface. This draining function improves safety and traction and reduces splash back during a storm event.

6.3 Rigid Pavements

6.3.1 Rigid Pavements and Layers

Rigid pavements consist of a properly prepared sub-grade, a sub-base or base layer, and a Portland cement concrete slab, the thickness of which is determined from the existing geotechnical and environmental conditions, and the anticipated loading it will experience during its service life.

The hyperlink below illustrates those basic components and layers of a rigid pavement structure.

http://www.pavement.com/pavtech/tech/fundamentals/main.html

6.3.2 Response to Loading

Rigid pavements respond to loading quite differently than flexible pavements. They generally do not flex under the actions of traffic loading. They resist applied loadings by bending of the concrete slabs. This "slab action" distributes the applied loads over a wider area of the base and sub-grade.

A typical rigid pavement response to loading is illustrated in the hyperlink below. It is also compared to that of an asphalt pavement.

http://www.pavement.com/pavtech/tech/fundamentals/fundasph.html

6.3.3 Load Transfer

Dowel bars are used to mechanically connect adjacent slabs without restricting horizontal joint movement. They function to assist in the load transfer, as well as assure a monolithic behavior of adjacent slabs. Dowel bars also reduce the slab deflection, and minimize faulting, thereby reducing stresses developed in the slab from loading. The hyperlink below discusses in greater detail the benefits of using dowels for load transfer.

http://www.pavement.com/pavtech/tech/fundamentals/fundloadtran.html

6.3.4 Widened Slabs

Widened slabs also assist in improving the load carrying capability. The benefit of added slab width reduces vertical deflections thereby reducing stresses (tensile) at the extreme fibers of the concrete. Typically a widened slab for mainline paving is 14 feet wide. Slabs wider than 14 feet may be used for ramp construction. This single slab width typically does not exceed 16 feet. Wider slab widths were used and have failed near mid-slab.

6.3.5 Types of Rigid Pavements Used by GDOT

Full depth rigid pavement types, used by the Department nowadays include the following types of pavements:

- **Jointed Portland Cement Concrete Pavements** (JPCP), are pavements containing enough joints to control all natural cracks expected in the concrete.
    - Steel tie bars are generally used at longitudinal joints to prevent joint opening.
    - Dowel bars are plain steel bars that are 1 ½ inches in diameter that assist in load transfer between adjacent slabs at planned and evenly spaced transverse contraction joints in the pavement.
    - GDOT specifies a 15 foot joint spacing for Interstates and higher duty facilities, and 20 feet elsewhere.

- **Continuously Reinforced Concrete Pavements** (CRCP), are pavements with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints.

- **Un-bonded Concrete Overlays** consist of a new concrete overlay of an existing concrete pavement or a composite pavement. Prior to placing the new concrete overlay, a bond breaker layer shall be placed to separate the new concrete from the surface that is being rehabilitated. Overlays re-use the existing pavement as a base and minimize the disturbance of the subgrade:
  - Of an existing Portland Cement Concrete Pavement
  - Of a Portland Cement Concrete Pavement that has already been overlaid with an asphalt concrete surface

The following rigid pavements are thinner than conventional full depth rigid pavements. They have recently been used by the Department on intersection improvement rehabilitation projects.

- **Conventional Whitetopping** is a new concrete overlay that ranges from 4 inches to 8 inches in thickness. It is placed directly onto an existing distressed asphalt pavement for rehabilitation purposes, with no particular steps taken to ensure bonding or de-bonding to the underlying pavement or substrate.
- **Ultra Thin Whitetopping** (UTW) is an asphalt pavement rehabilitation method that uses a thin layer of high strength concrete with the depth of rehabilitation between 2 and 4 inches. The remaining asphalt concrete pavement should be in relatively good condition, adequate in thickness (> 3 inches), and the key to a successful UTW is to ensure the bonding of the concrete to the underlying asphalt.

In order that whitetopping pavement types are considered, the following conditions should exist:

- The asphalt concrete surface should be rutted so that reconstruction or partial depth reconstruction is warranted.
- The rutting is normally due to high traffic volumes.
- The rutting is also due to the high turning movements at that specific location, and with intersections in general.
- After milling, an adequate depth of asphalt layer, in good condition, shall remain in place.

References:

1. AASHTO Interim Guide for Design of Pavements 1972, American Association of State Highway and Transportation Officials. Washington, D.C., 1974.
2. NCHRP Synthesis 342, Chip Seal Best Practices, Transportation Research Board, Washington, D.C., 2003
3. Pavement Preservation Glossary of Terms, Foundation for Pavement Preservation, Austin, Texas, 2001
4. Glossary of Terms, American Concrete Pavement Association, Skokie, Illinois, 2005
5. Asphalt Industry Glossary of Terms, The Asphalt Institute, Lexington, KY, 2004

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7 Loads and Stresses Applied to Pavements

7.1 Traffic Analysis

Traffic analysis for pavement structure design is supplied by the Office of Environment and Location (OEL), Traffic Analysis Section. The designer should request traffic diagrams from this unit for the project on hand. If the traffic analysis was conducted by a consultant, it has to be approved by the OEL before given further consideration.

Check the base year (opening day) against the Department's projected let date and what you determine, based on engineering judgment and input from other offices, may be the realistic let date. Use the later estimated let date, add the number of years for construction (usually 2 years) to determine your base year (opening day), and add 20 years for the design year. Develop your traffic projections based on counts or the traffic diagrams provided by Office of Environment and Location. Traffic diagrams should be for both ADT (base and design year) and DHV (a.m. and p.m. design year). All new and revised traffic projections must be approved through the Office of Environment and Location, Traffic Analysis Section. See sample traffic diagrams in Figures 7.1 and 7.2.

Twenty-four hours truck percentage should also be developed as part of the traffic diagram, and shown as a breakdown between Single Units (SU), and combination or Multiple Units (COMB or MU). Adjustments for directional distribution and lane distribution will be made by the OEL Traffic Analysis Section or, if desired, the unadjusted data can be obtained and the distribution percentages provided. The traffic data figures to be incorporated into the design procedure are in the form of 18 kip equivalent single axle load applications (see section 7.2 for additional discussion).

7.2 Pavement Loading - Estimating Design ESALs

The procedure to predict the design ESALs is to convert each expected axle load into an equivalent number of 18k ESALs and to sum these over the design period. Thus, a mixed traffic stream of different axle loads and axle configurations is converted into a number of 18k ESALs. The following steps are used by GDOT to calculate ESALs:

1. Determine the Average Daily Traffic (ADT) for the opening year and the last year of the design period. Use the appropriate growth factor from the GDOT/OEL Traffic Analysis Unit. Calculate the average ADT for the design period by adding the ADT of the first year and the ADT of the last year and dividing by two.
2. At the present time three classifications are used: Passenger cars and pickup trucks; Single Unit trucks; and Multi-Unit or combination trucks.

3. Multiply the number of vehicles in each classification by the appropriate 18k equivalency factor. The damaging effect of an axle is different for a flexible pavement and a rigid pavement, therefore there are different equivalency numbers for the two pavements. **Table 7.1** lists the statewide equivalency factors that are currently used in pavement design. Those factors are based on weigh-in-motion data dating back to the late 1980's.

Vehicle Classification	**Pavement Type**	
	Flexible Pavement	Rigid Pavement
Passenger cars & pickup trucks[1]	0.004	0.004
Single unit trucks	0.400	0.500
Combination trucks	1.500	2.68

TABLE 7.1: GDOT EQUIVALENCY FACTORS

[1] APD Software does not account for passenger cars and pickup trucks for flexible pavement.

Note: Add the product of each equivalency factor and number of vehicles to yield a single total number of equivalent 18-kip ESALs for the pavement type being designed.

4. Multiply this number by 365 (days in a year) and the number of years in the design period. This number is the total 18k ESALs for the roadway.
5. Multiply the total 18k ESALs for the roadway by the lane distribution factor (LDF) in Table 7.2 below or use Appendix A which relates LDF to volumes and the number of lanes.

Facility	LDF[1](in percent)
Four lane Rural Freeway	85-100
Four Lane Urban Freeway	60-80
Six Lane Rural Freeway	70
Six Lane Urban Freeway	60
Six Lane Rural Highway Free Access	70-100
Six Lane Urban Highway -Free Access	60-80
Two Lane Highway and Ramps	100

TABLE 7.2: LANE DIST. FACTORS TO CONVERT TOTAL 18K ESAL TO DESIGN LANE 18K ESAL

Note: Steps 1-5 above reflect the procedure for a rigid pavement design (see Chapter 11.5.1). In a flexible pavement design ESALs are determined within the APD software. When using WINAPD to compute design ESALs enter LDF as a whole number rather than a percentage (see Chapter 11.4.1).

7.3 AASHTO Design Method and ESAL Estimation

The 1986-1993 AASHTO Guide incorporates many modifications to the pavement design procedures for both concrete and asphalt procedures; although the basic design models for both remained the same as in previous versions. There are principal modifications to the AASHTO pavement design methodology in the 1986-1993 procedure when compared to the AASHTO 1972 guide.

7.3.1 Rigid Pavement

- Addition of a drainage adjustment factor;
- Addition of a multiplier for pavement thickness that presumably is less than 1.0 for drainage conditions worse than those in the AASHO Road Test and greater than 1.0 for better drainage conditions;
- Determination of the design k-value as a function of the subgrade resilient modulus, depth to a rigid layer, base thickness and elastic modulus, erodability of the base material, and seasonal variation in soil support;
- Presentation of corner stress adjustment (J-factor) values as a function of pavement type (jointed or CRCP), load transfer (doweled or aggregate interlock), and shoulder type (asphalt or tied concrete).

7.3.2 Flexible Pavement

The following are changes in flexible pavement:

- The soil support value has been replaced with resilient modulus (M_r)
- The structural number (SN) has been modified by addition of drainage coefficients
- The Regional factor has been deleted.

7.3.3 Other Modifications

A reliability adjustment factor is applied to the design ESAL input instead of using a factor of safety on the modulus of rupture. This factor reflects the degree of risk of premature failure that the agency is willing to accept. Facilities of higher functional classes and higher traffic volumes warrant higher reliability adjustment factors in design. The magnitude of the adjustment is a function of the overall standard deviation associated with the AASHTO model, which reflects the following:

- Errors associated with estimation of each of the inputs (ESALs, subgrade *k*, concrete strength, serviceability, etc.)
- Errors associated with the quality of fit of the model to the data on which it is based

- Replication errors (differences in performance of seemingly identical pavement sections under identical conditions). When reliability adjustments are made to the traffic input in this manner, average values should be used for the material inputs (*k*-value, M_r, *E*); that is, no other safety factors should be applied to any of these inputs.

Both rigid and flexible equations have been modified to consider total serviceability loss.

7.3.4 Design ESAL

The major difference in the way that GDOT and AASHTO compute Design ESALs is in the following two factors:

- Axle load equivalency factors are a function of pavement type, thickness or structural number, terminal serviceability, and other factors.
- The lane distribution factor varies with the volume of traffic and the number of lanes as indicated in Appendix A.

7.4 Loads and Stress Calculations on Asphaltic Pavement Structures

7.4.1 Loads on Flexible Pavements

Traffic loadings are the vehicle forces exerted on the pavement. Because one of the primary functions of the pavement is to distribute loads, pavement design must account for expected lifetime traffic loading. Loads can be characterized by tire loads, axle and tire configurations, and load repetition.

- **Tire Loads** - Tire loads are the fundamental loads at the actual tire-pavement contact points.
- **Axle and Tire Configurations** - While the tire contact pressure and area are of concern, the number of contact points per vehicle and their spacing is critical. As tire loads get closer together, their influence areas on the pavement begin to overlap. When this begins to occur the design characteristic of concern is no longer the single isolated tire load but rather the combined effect of all the interacting tire loads.
- **Load Repetition** - Loads, along with the environment, damage pavement over time. The standard model asserts that each individual load inflicts a certain amount of unrecoverable damage. This damage is accumulated over the life of the pavement until it reaches some value when the pavement is considered to have reached the end of its useful service life.

7.4.2 Environment Loading

A pavement must function within its environment. Environmental variations can have a significant impact on pavement materials and the underlying subgrade, which in turn can drastically affect pavement performance. Key environmental factors of concern in Georgia are typically temperature and moisture.

Temperature

Temperature acts on pavements in two principal ways:

- Temperature extremes can affect asphalt binder rheology. Asphalt binder rheology (deformation and flow characteristics) varies with temperature. Therefore, estimated temperature extremes and their effects are a primary consideration when selecting an appropriate asphalt binder. Older asphalt binder grading (Viscosity and Penetration Grading) systems did not directly account for temperature effects, and thus various empirical systems and thumb-rules were developed. The Superpave PG binder grading system corrects this deficiency by grading asphalt binder based on its performance in relation to temperature.
- Temperature variations can cause pavement to expand and contract. Pavements, like all other materials, will expand as they rise in temperature and contract as they fall in temperature. Small amounts of expansion and contraction are typically accommodated without excessive damage; however, extreme temperature variations can lead to catastrophic failures. Flexible pavements in colder areas can suffer transverse cracks as a result of excessive contraction in cold weather. In Georgia, this cold weather contraction is typically not enough to cause cracking.

Moisture

Moisture, in the form of accumulated water or rainfall, can affect pavement design and construction as well as basic driving conditions. Specific issues with moisture are:

- **Design** - When the design engineer is aware of potential water problems that are reported in the Soil Survey Summary, then it is his responsibility to provide the appropriate materials and methods in the plans to keep the subgrade soil at its optimum moisture content during and after construction.
- **Construction** - The project engineer should insure that:

The subgrade is compacted to its optimum moisture content and 100% of its laboratory maximum dry density.

Any planned underdrain systems are installed.

Any potential water problems that were not reported in the Soil Survey Summary are addressed.

HMA should not be placed in wet conditions because excessive water may damage the hot, fresh HMA by cooling it too quickly or getting into the mix and causing later stripping problems.

Driving Conditions - Rainfall reduces skid resistance and can cause hydroplaning in severely rutted areas.

7.5 Responses of Flexible Pavements Under Load

7.5.1 Stress

The stresses that occur in a HMA pavement under load are quite complex; routine calculation of these stresses is a recent development and is not presented in this manual.

7.5.2 Deflection

HMA pavements are often described as "flexible" because they deflect under load. Pavement deflections represent an overall "system response" of the pavement structure and subgrade soil to an applied load. When a load is applied at the surface, all layers deflect, creating stresses and strains in each layer, as illustrated in Figure 7.3. For HMA pavements, the critical pavement responses under a wheel load are the following:

- Maximum deflections immediately beneath the wheel load
- Tensile strain at the bottom of the HMA surface and asphalt-treated base layers
- Vertical strain in the base/subbase layers
- Vertical strain at the top of subgrade soil

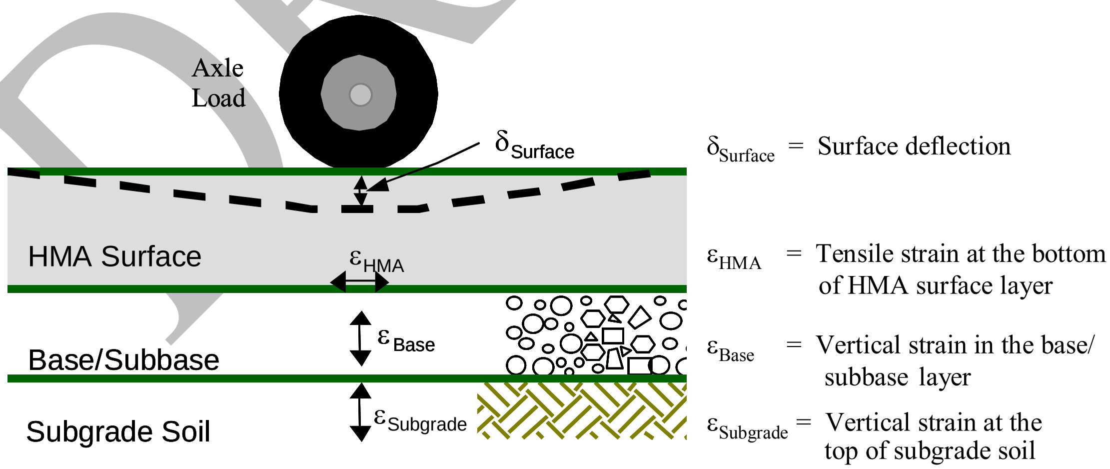

FIGURE 7.3 - ILLUSTRATION OF HMA PAVEMENT RESPONSES TO A WHEEL LOAD

Figure 7.4 illustrates the effects of the "strength" of a pavement structure using a deflection profile. As shown in this figure, the deflection profile reflects the structural capacity of the pavement. A "stronger" pavement exhibits a flatter deflection profile, because it is able to spread the load to a larger area. The deflection profile also reflects the stiffness of the pavement structure relative to subgrade soil stiffness. These relationships can be used to back-calculate the moduli values of each pavement layer and the subgrade soil.

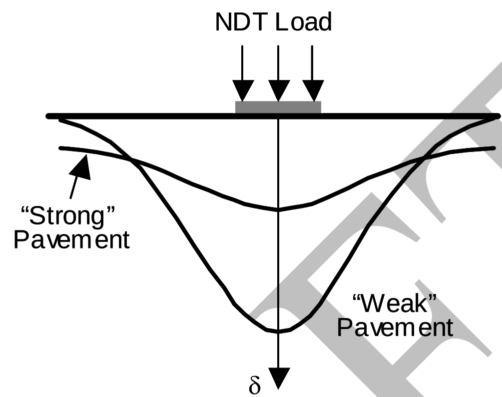

FIGURE 7.4. ILLUSTRATION OF THE EFFECTS OF PAVEMENT STRUCTURE ON DEFLECTION PROFILE

7.6 Loads and Stress Calculations on Rigid Pavement Structures

7.6.1 Basis of Rigid Pavement Design

The AASHTO Guide for Design of Pavement Structures (AASHTO Guide for Design of Pavement Structures – 1981 revision, American Association of State Highway and Transportation Officials. Washington, D.C. 2001) is the only approved design method for rigid pavements for GDOT. The basis for the design of any pavement structure is its ability to carry the intended loading over its design period. In rigid pavements, this would be the necessary slab thickness required to carry the lifetime loading. This thickness is a function of the following parameters:

- Traffic Loading Volumes over the design period; such as the volumes of the base year and terminal year
- Traffic Loading Composition during the design period; such as the percent traffic mix composition of Multi-Unit, Single-Unit and Personal Vehicles as percentages (totaling to 100).

- The Modulus of Rupture f_r of the concrete (flexural strength), is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. A f_r of 600 psi at 28 days should be used with the current statewide specification for concrete pavement design. If an alternate value for f_r, then it must be explained and documented with laboratory test data, and
- The Effective Modulus of Subgrade Reaction, k_{eff}, allows pavement designers to take into account the structural benefits of all layers under the concrete slab.

7.6.2 Rigid Pavement Response to Loading

Rigid pavements respond to loading in a variety of ways that affect performance (both initial and long-term). The three principal responses are:

- **Curling stress** - Differences in temperature between the top and bottom surfaces of a PCC slab will cause the slab to curl. Since slab weight and contact with the base restrict its movement, stresses are created.
- **Load stress -** Loads on a PCC slab will create both compressive and tensile stresses within the slab and any adjacent one (as long as load transfer efficiency is > 0).
- **Shrinkage/Expansion -** In addition to curling, environmental temperatures will cause PCC slabs to expand (when hot) and contract (when cool), which causes joint movement.

These three principal responses typically determine PCC slab geometry (typically described by slab thickness and joint design). As slabs get longer, wider and thinner, these responses, or a combination of them, will eventually exceed the slab's capacity and cause failure in the form of slab cracking, joint widening or blowup. Note that additional issues, notably load transfer stresses and deflections, must also be accounted for in design.

There are a variety of ways to calculate or at least account for these responses in design. The empirical approach uses the AASHO Road Test results to correlate measurable parameters (such as slab depth and PCC modulus of rupture) and derived indices (such as the load transfer coefficient and pavement serviceability index) to pavement performance. The mechanistic-empirical approach relates calculated pavement stresses to empirically derived failure conditions.

7.6.3 Rigid Pavement Stresses

The stresses of primary concern are associated with slab bending either due to temperature gradients, loading or a combination thereof.

Since PCC is much stronger in compression than tension, tensile stresses control PCC pavement design.

7.6.4 Loading Stresses

The critical load induced stresses that are designed for in a rigid jointed pavement structure are the tensile stresses that occur at the slab bottom and at slab mid-span where the deflection due to loading is maximum. Those stresses are tensile stresses (bottom of slab extending, top of slab is compressing)

Tensile stresses due to axle loading are generated when the wheels are tangent to the unsupported edge of the pavement slab. They are proportional to the slab deflections. At transverse joints, and if load transfer devices, such as dowel bars are used, slab deflections are reduced, and the tensile loading become less severe than those at slab mid-span where the deflections are maximum.

In CRC pavements the longitudinal reinforcing steel provides continuity of load carrying capacity. Therefore, unlike a Jointed PCC pavement, there is no critical location for stress computations.

Any location within the CRC Pavement deflects by the same amount in response to a given axle load. Therefore the tensile stresses generated by the axle loadings are independent of location along the pavement.

7.6.5 Loading Stress Calculations

The following figures represent original equations developed by Westergaard (1926) for three critical load locations are (after Bradbury, 1938 and Westergaard, 1926). Assuming a poison's ratio = 0.15:

- Interior loading - Occurs when a load is applied on the interior of a slab surface which is "remote" from all edges.

Interior loading (tensile stress at the slab bottom)

$$\sigma_i = \frac{0.3162(W)}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+1.069\right]$$ Equation 1

- Edge loading - Occurs when a load is applied on a slab edge "remote" from a slab corner.

Edge loading (tensile stress at the slab bottom)

$$\sigma_e = \frac{0.572(W)}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.359\right]$$ …Equation 2

- Corner loading - Occurs when the center of a load is located on the bisector of the corner angle.

Corner loading (tensile stress at slab top)

$$\sigma_c = \frac{3(W)}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right]$$ Equation 3

where:	σ_i, σ_e, σ_c	=	maximum stress (psi) for in interior, edge and corner loadings, respectively
	W	=	wheel load (lbs.)
	h	=	slab thickness (inches)
	a	=	radius of wheel contact area (inches)
	l	=	radius of relative stiffness (inches)
	b	=	radius of resisting section (inches) $= \sqrt{1.6(a^2) + h^2} - 0.675(h)$

Note: All three equations involved the depth of slab (h) *squared*. This suggests that slab thickness is very critical in reducing load stresses to acceptable levels.

7.6.6 Thermal and Curling Stresses

Slab curling and thermal stress calculations seek to find the points of maximum tensile stress as the slab curls due to internal temperature gradients (see Figure 7.5 and Figure 7.6 below).

In 1935, measurements reported by Teller and Southerland of the Bureau of Public Roads showed that the maximum temperature differential (hence, maximum curling and maximum tensile stresses) is much larger during the day than during the night. Therefore, the daytime curling stresses are usually the limiting ones to be considered for design purposes.

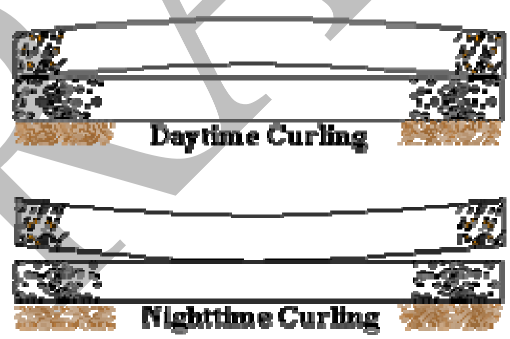

FIGURE 7.5 SLAB CURLING

Daily temperature fluctuations induce thermal stresses in slabs as a result of expansion and contraction of the slab surfaces.

When the slab surface temperature rises such as during a summer day, its surface temperature becomes greater than the temperature at its bottom. This temperature differential Δt causes the slab surface to expand in relation to its bottom. This temperature differential induces thermal stresses.

The opposite happens when the surface temperature drops in relation to the slab bottom. This temperature differential Δt causes the slab bottom to expand relative to the slab top. This fluctuation, due to temperature change alone induces stresses in the slabs.

Note: To evaluate the tensile warping stresses which develop in the slab, the temperatures at the top and bottom of the slab must be estimated.

For interior stresses, Bradbury's formula is the following:

$$\sigma_t = \frac{(E)(e)(\Delta T)}{2}\left[\frac{C_1 + \mu C_2}{1-\mu^2}\right]$$ …Equation 4

where: σ_t = slab interior warping stress
E = modulus of elasticity of PCC
e = thermal coefficient of PCC (0.000005/°F)
ΔT = temperature differential between the top and bottom of the slab
C_1 = coefficient in direction of calculated stress
C_2 = coefficient in direction perpendicular to C1
μ = Poisson's ratio for PCC (0.15)

For Edge stress at the midspan of the slab:

$$\sigma_t = \frac{(C)(E)(e)(\Delta T)}{2} \quad \text{Equation 5}$$

where: σ_t = slab edge warping stress
C = coefficient which is a function of slab length and the radius of relative stiffness (shown in Figure 7.7)
E = modulus of elasticity of PCC
e = thermal coefficient of PCC (0.000005/°F)
ΔT = temperature differential between the top and bottom of the slab

Bradbury also developed the following approximate formula for slab corner warping stresses.

$$\sigma_t = \frac{(E)(e)(\Delta T)}{3(1-\mu)}\left[\sqrt{\frac{a}{l}}\right] \quad \text{Equation 6}$$

where: σ_t = slab interior warping stress
E = modulus of elasticity of PCC
e = thermal coefficient of PCC (0.000005/°F)
ΔT = temperature differential between the top and bottom of the slab
μ = Poisson's ratio for PCC (0.15)
a = radius of wheel load distribution for corner loading
l = radius of relative stiffness

The radius of relative stiffness (the relative stiffness of the slab relative to that of the foundation) is required for the above formulae. This equation is (from Westergaard, 1926):

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \quad \text{Equation 7}$$

where: l = radius of relative stiffness
E = modulus of elasticity of PCC
h = slab thickness
k = modulus of subgrade reaction
μ = Poisson's ratio for PCC (0.15)

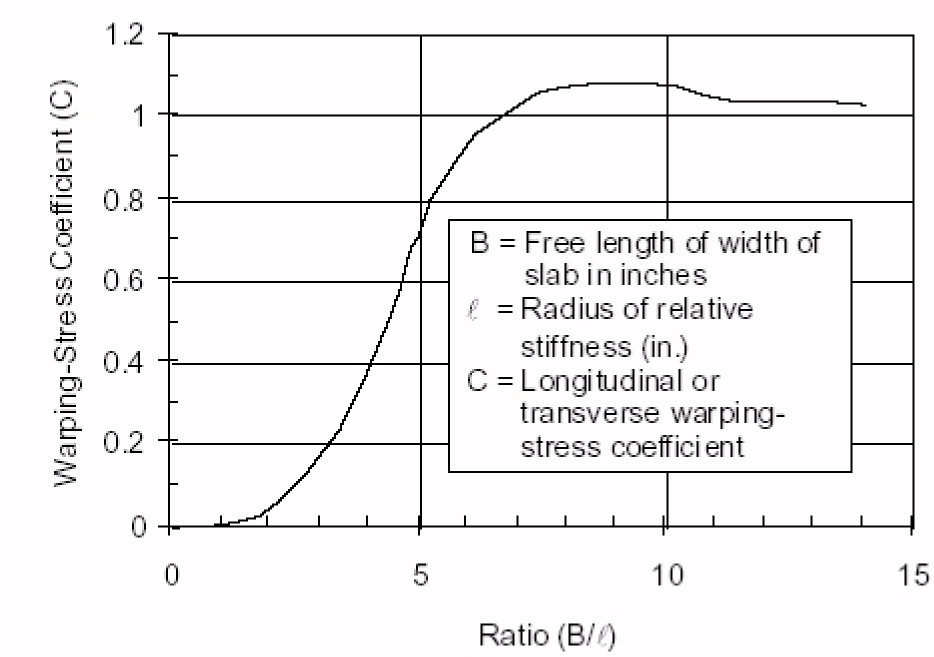

FIGURE 7.7: STRESS CORRECTION FACTOR FOR FINITE SLAB (BRADBURY, 1938)

7.6.7 Shrinkage/Expansion

Although slab shrinkage and expansion causes internal stress, especially as the PCC sets and hardens, the long term concern centers on the joint movement that this shrinkage/expansion can cause. The following formula can be used to estimate joint movement in PCC slabs (FHWA, 1989):

$$z = (C)(L)[(e)(\Delta t) + \delta] \quad \text{Equation 8}$$

where:

where:	z	=	joint opening = change in slab length (inches)
	C	=	base/slab frictional restraint factor
			= 0.65 for stabilized bases
			= 0.80 for granular bases
	L	=	slab length (inches)
	e	=	thermal coefficient of PCC (listed by coarse aggregate type)
		=	6.6×10^{-6}/°F (quartz)
		=	6.5×10^{-6}/°F (sandstone)
		=	6.0×10^{-6}/°F (gravel)
		=	5.3×10^{-6}/°F (granite)
		=	4.8×10^{-6}/°F (basalt)
		=	3.8×10^{-6}/°F (limestone)
	ΔT	=	the maximum temperature range (for some cases it is the temperature of the PCC at the time of placement minus the average daily minimum temperature in January) (°F)
	δ	=	shrinkage coefficient of PCC
		~	0.0008 in./in. for indirect tensile strength of 300 psi or less
		~	0.00045 in./in. for indirect tensile strength of 500 psi
		~	0.0002 in./in. for indirect tensile strength of 700 psi or greater

(Note: δ should be omitted for rehabilitation projects as shrinkage (assuming no new slab PCC) is not a factor.)

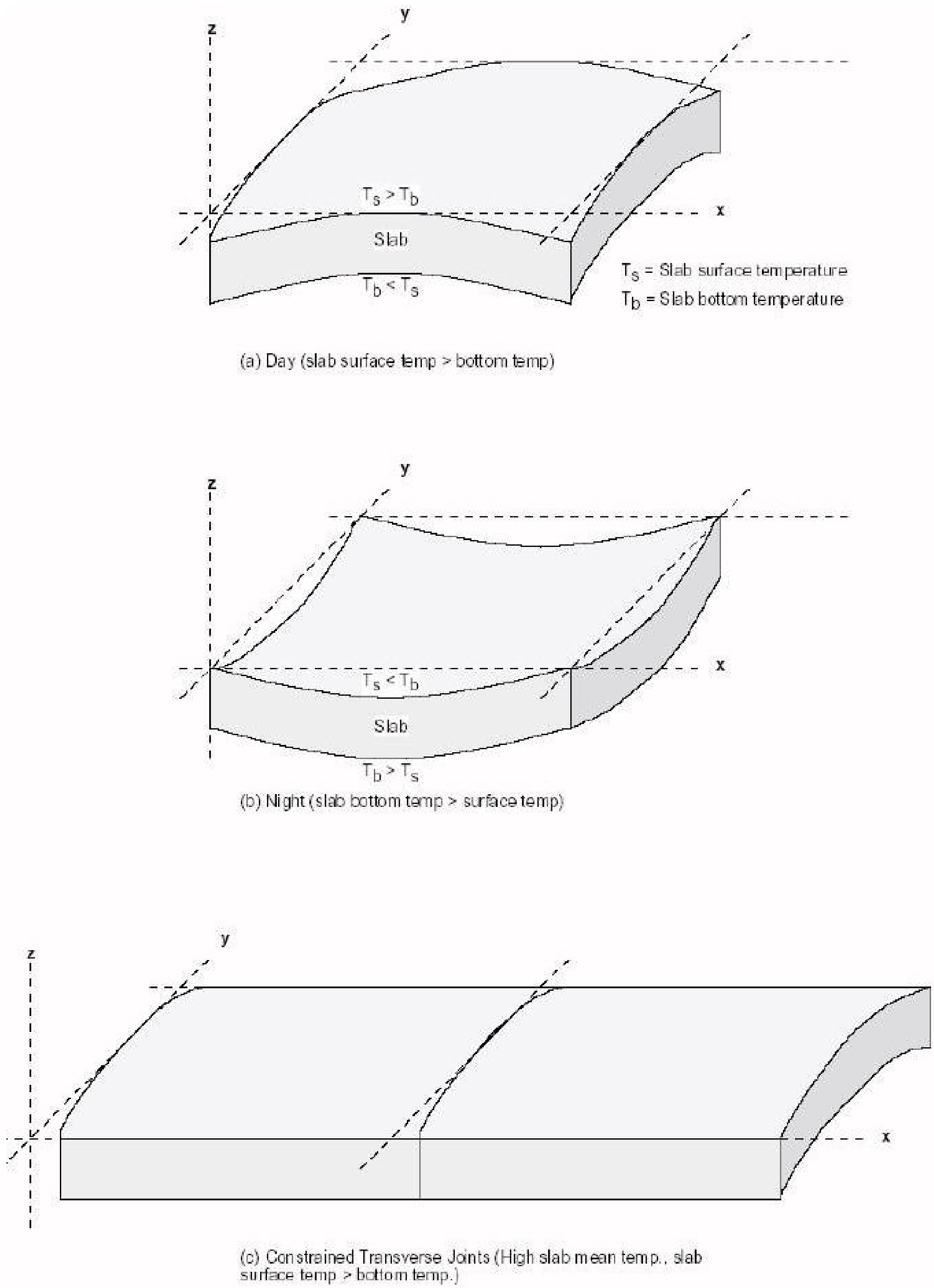

FIGURE 7.6 SLAB CURLING

References:

1. Technical Brief on Joint Spacing for JPCP (Abstract), American Highway Technology, 1999
2. Washington State DOT Interactive Pavement Design Guide

DAWSON COUNTY

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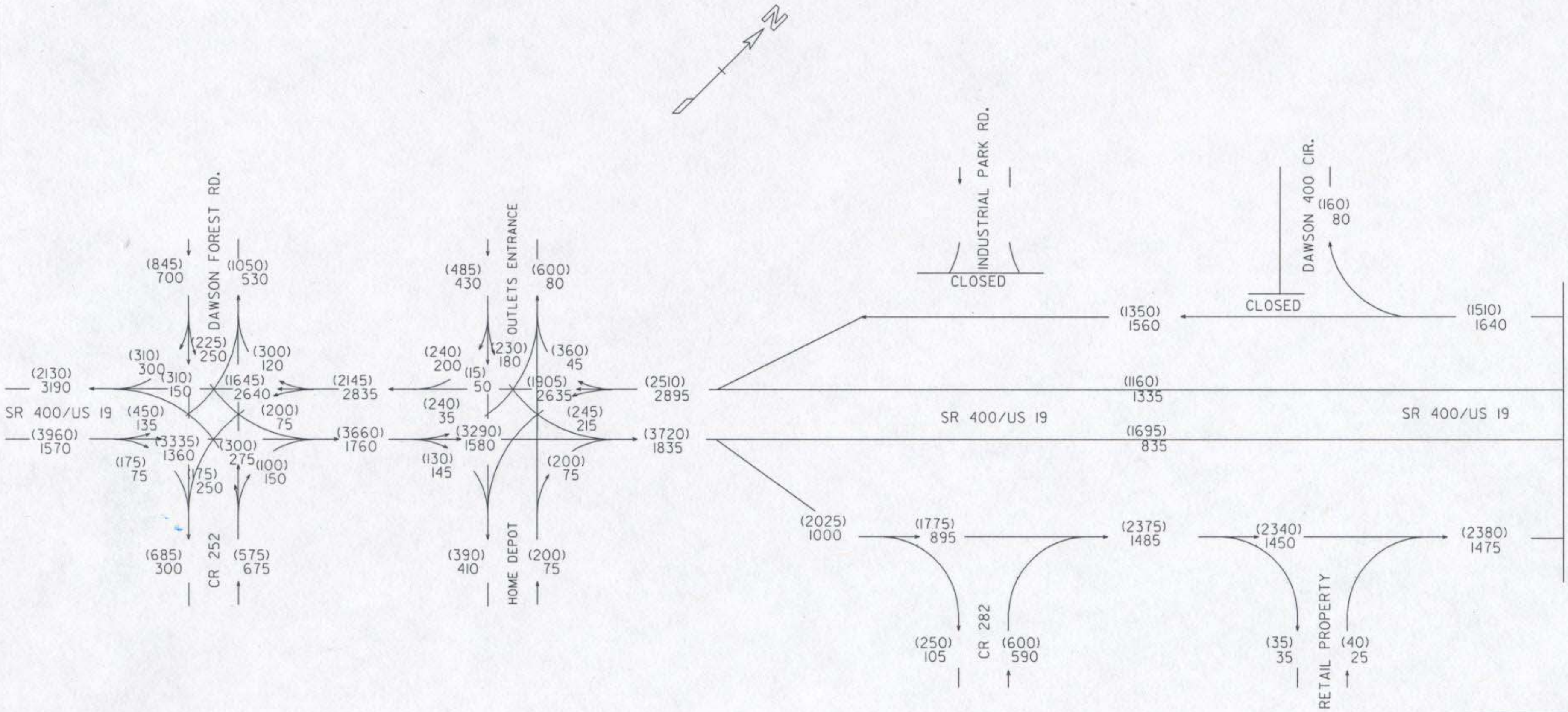

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DAWSON CO.

SR 400 FM CR 252/
DAWSON FOREST RD.
TO CR 128/
KILOUGH CH.RD.

2028 PM DHV = (000)
2028 AM DHV = 000

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FIGURE 7.1

TJW
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OFFICE OF ENVIRONMENT/LOCATION

DAWSON COUNTY

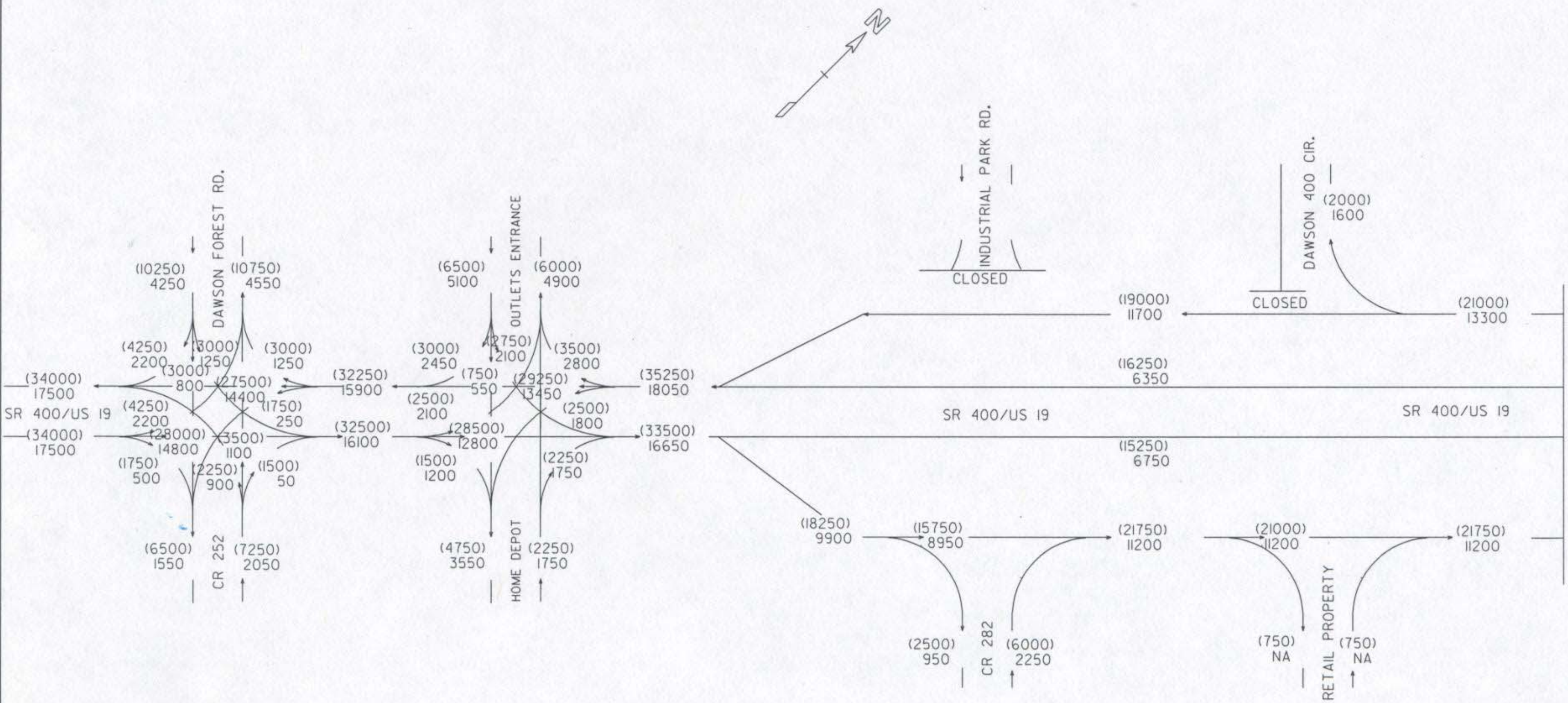

FIGURE 7.2

APD-056-1(63)
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DAWSON FOREST RD.
TO CR 128/
KILOUGH CH.RD.

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8 Measuring Pavement Distresses

8.1 Flexible Pavement Distresses (PACES)

The Pavement Condition Evaluation Survey (PACES) is a manual that is used to objectively rate existent flexible pavement statewide. COPACES is the computerized version of PACES. The system is maintained by the Office of Maintenance within GDOT. It is designed to indicate the amount and type of surface distress on a roadway at the time the survey is made. A number of distresses have been identified for flexible pavement and surface treatment which relate to the performance of the pavement. These distresses are as follows:

- Rut Depth
- Raveling
- Load Cracking
- Edge Distress
- Block Cracking
- Bleeding/Flushing
- Reflection Cracking
- Corrugations/Pushing
- Patches and Potholes
- Loss of Section

For more in depth discussion on PACES, refer to Appendix E

8.2 Rigid Pavement Distresses (CPACES)

The Concrete Pavement Condition Evaluation Survey (CPACES) is a manual that is used to objectively rate Jointed Plain Concrete Pavement (JPCP) statewide. The system is maintained by the Office of Maintenance within GDOT and is not yet fully implemented as PACES. It is designed to indicate the amount and type of surface distress on a roadway at the time the survey is made.

A number of distresses have been identified for the JPCP that relate to the performance of the pavement. These distresses are the following:

- Faulting
- Broken Slabs
- Slabs with Longitudinal Cracks
- Replaced Slabs
- Failed Replaced Slabs
- Joint Defects
    - Joints with spalls
    - Joints with patched spalls
    - Joints with failed spall patches
- Shoulder Joint Distress

For more in depth discussion on CPACES, refer to Appendix F.

8.3 Other Distresses

There are other types of concrete pavement distresses which are not considered in CPACES either because they occur infrequently or they are included in one of the CPACES distress categories at a certain severity level.

They are included in Appendix F to provide a general pavement survey overview. They include; punch-out, polish aggregate and Scaling, D cracking, Map cracking, blow-ups, water bleeding and pumping and pop-outs.

For more in depth discussion on these distresses, refer to Appendix F.

9 Existing Pavement Evaluation

An existing pavement evaluation will aid in determining whether an overlay pavement design is acceptable, and will be of great benefit early in project concept and development. There are two main project phases to consider when developing an existing pavement evaluation: concept or project scope, and project design/plans development.

9.1 Concept or Project Scope

Listed below are information sources and items to consider when developing project concept or determining project scope:

- **Project History:** Research the history of the project.
- **Original Plans:** Check the original plans for the existing section. Original plans may be found in the Plans File Room in the Office of Road Design. The old plans/files may be either on micro-film or maintained as .tiff image files. At this time the only access is through the Plans File Room micro-film viewers or GDOT personal computers for the .tiff images. Some Districts also maintain plans/files of their projects and as-builts, so do not discount this as a source also.
- **Adjoining Projects:** Are there any adjoining projects? If so, what typical section and materials were used?
- **Specific Local Characteristics:** Are there any specific local characteristics? For example, is there an area in which a large concentration of tractor trailers is present, perhaps at a warehouse. Or, are the trucks in the vicinity of the project loaded heavier than the norm possibly at a cement plant or rock quarry?
- **Maintenance History:** Has this site had a history of maintenance problems? How many times has the existing pavement been overlaid and what were the depths?
- **Railroad Considerations:** Does the project parallel a railroad, and offsetting from railroad right of way is required to the point that the existing cannot be utilized?
- **Section Retentions:** Are there so many substandard vertical and horizontal curves or situations where the widening may alternate left and right to the point that sections that can be retained are no longer than 1000 feet? If so, discuss with District Construction whether retaining existing is worthwhile.
- **Preliminary Pavement Evaluation Report:** A Preliminary Pavement Evaluation Report is required during Concept Development/Validation, Phase I of the project. Conduct a "Windshield Survey" to develop a good understanding of the existing pavement, shoulders, ditches, and so on. Obtain preliminary values for soil support from Appendix G and regional factor from Appendix H. Perform preliminary pavement designs (Chapter 11) utilizing project design traffic information. The report should be submitted in PACES (Appendix E) or CPACES (Appendix F) format, and also attached to the Concept or Concept Validation Report.

9.2 Project Design and Plans Development

During project design and plan development all items in section 9.1 should be revisited. The following items will assist in providing greater detailed information for developing and determining if an overlay pavement design will be preferred:

- **Existing Pavement Condition Evaluation (E.P.E.):** Request an E.P.E. for the project. See Appendix C.
- **Section 9.1:** All of the items in Section 9.1 should be revisited verifying the history had been checked and was addressed appropriately.
- **DOT Office Involvement:** The DOT Office involved in the project development and overview of the project would be the Office submitting the request, plans and correspondence to OMR. If a consultant project, then this process is between the Prime and sub, with recommendations submitted to OMR for review, comments, and approval.
- **Project Types:** The project could be a bridge replacement or widening, the addition of passing lanes, an upcoming maintenance project, or a long range project, in short any project involving a pavement section.
- **Plan Specifications:** Two sets of half-size plans are needed including cover, typical sections, plan and profile, and earthwork cross sections. Specific limits of pavement proposed to be retained should be clearly marked in the plans, with mile post numbers.
- **Traffic Information:** This data should include as a minimum the following items: Base Year and Design Year, 24 hour % Trucks, Multi-axle Units, Single-axle Units. The 24 hour truck percentage is a very sensitive variable and may alter designs considerably.
- **Pavement Retention:** What percent of the existing pavement is being retained. Will the wheel path be directly over a joint? If so, removal of pavement in wheel path may be desirable or fabric spanning the joint might be an option.
- **Bridge Clearance:** When retaining existing pavements, bridge vertical clearances must be checked to make sure they are adequate.
- **Evaluation Timeline:** Allow 9 months for the evaluation to be returned, if submitted to OMR to initiate and complete.
- **Miscellaneous Pavement Information:** The lab (if submitted to OMR) or Prime/Sub Consultant will provide information as to the existing pavement's quality, rutting, structural composition, joint repair, and several other recommendations. The Prime/Sub Consultant should provide this information in PACES or CPACES format.
- **Soil Survey:** Generally, the request for evaluations would occur simultaneously with the request for a Soil Survey.
    - EPEs will have a ***recommended*** pavement structure with a GAB Base alternate only. Base alternates, as well as minimum base thickness requirements, are recommended in the approved Soil Survey Summary.

10 Pavement Type Selection Process

Pavement Type Selection is a process by which the most effective pavement type is determined for a specific project or a planned corridor, considering engineering, economic, and other factors. It involves the following:

- Combines engineering and economic analyses.
- Provides data that assists engineers in choosing a cost-effective pavement type.
- Is not an exact science. The 1993 AASHTO Guide recognizes this fact and allows for other factors, major and minor, that need to be considered along with engineering and economic factors.
- Is a new process for the GDOT and like all new processes it is subject to refinements and revisions.

At the conclusion of a Pavement Type Selection for a selected project or planned corridor, the pavement being selected, regardless of type shall:

- Be capable of carrying the anticipated loading during the design lifetime.
- Be capable of performing under site specific geotechnical (soil support) and environmental (precipitation and drainage) conditions.

10.1 Pavement Design

Pavement Design is the process of selecting a combination of materials of known strengths and thickness that are able to withstand and support the anticipated lifetime loading repetitions.

The pavement is expected to perform under the site specific geotechnical, environmental, and traffic conditions.

10.2 Design Period

Design Period is the period of time from when the pavement is placed in service to the time the pavement is expected to deteriorate to its terminal serviceability level. GDOT uses a design period of 20 years for both rigid and flexible pavements.

10.3 Serviceability Loss

Serviceability loss is the gradual loss in pavement quality over its design life. GDOT uses an initial serviceability level of 4.5 and a terminal serviceability level of 2.5 (AASHTO 1972) for permanent pavements.

10.4 Analysis Period

Analysis Period is the length of time for which an LCCA is conducted for economic analysis of the various alternate pavement types under consideration.

10.5 New Construction Projects

New Construction Projects are construction projects intended to add new capacity to the entire network by adding new facilities.

10.6 Total Reconstruction Projects

Total Reconstruction Projects are construction projects in which the existing pavement has reached a terminal level of serviceability, or has reached a point of diminishing returns with any planned maintenance activity.

10.7 Rehabilitation Projects

Rehabilitation projects are construction projects in which the existing pavements are in need of some treatment or upgrade so as to restore the pavement to an acceptable level of servicability.

10.8 Widening Projects

Widening Projects are construction projects intended to add capacity to an existing facility.

10.9 Existing Pavement Evaluations

Existing Pavement Evaluations are needed when the existing pavement or portions thereof will be utilized in the proposed construction. GDOT's procedures and guidelines for requesting and performing Existing Pavement Evaluations are outlined in Chapter 9 and detailed in Appendix C.

10.10 Life Cycle Cost Analysis

LCCA is an analysis tool that compares alternate pavement types which are designed for a given project. LCCA compares the associated costs, including future maintenance and rehabilitation costs, over an Analysis Period for each alternate pavement type.

An LCCA analysis considers at least two viable alternate pavement types for consideration. LCCA also incorporates user costs as a result of construction, maintenance, and repair work for each proposed design alternate being evaluated.

Note: At the time of initial printing (2005), a LCCA will only be required at the direction of the PM or in the event of a VE study. In most cases, the LCCA will be performed by OMR or a qualified consultant as needed.

10.10.1 Introduction

This chapter provides information on Life Cycle Cost Analysis (LCCA) for pavement designs. Guidelines for when an LCCA is required are included. A discussion of deterministic and probabilistic life cycle cost analysis is included as well as typical analysis procedures, inputs, and evaluation of alternatives.

Life cycle cost analysis techniques are typically considered when making decisions regarding pavement type selection and determination of appropriate pavement design or pavement rehabilitation strategies. The pavement design alternative with the lowest life cycle cost will typically be the preferred alternative. However, when alternatives have comparable life cycle costs, other factors may be used to base a decision.

According to the September 1998 FHWA Interim Technical Bulletin entitled "Life Cycle Cost Analysis in Pavement Design - In Search of Better Investment Decisions", the FHWA position on LCCA is that it is a decision support tool, and the results of LCCA are not decisions in and of themselves. The FHWA encourages the use of LCCA in analyzing all major investment decisions where such analyses are likely to increase the efficiency and effectiveness of investment decisions.

LCCA are required as part of a “Value Engineering Study”. Value Engineering (VE) Studies are required on federal-aided projects on the National Highway System with a total estimated cost of $25 million (including PE, ROW, Utilities and Construction) or over. See TOPPS 2450-1 for guidance on VE studies.

10.10.2 Projects Requiring LCCA

At this time GDOT’S practice is that all new freeway ramps should be PCC and PCC should be strongly considered for heavily trafficked arterials such as interstates and bypasses. PCC should also be considered as an alternate on other projects if the project (or portion of) is new location, existing pavement is to be totally removed, a section of mainline between ramp terminals can be constructed with PCC, or ADT and truck percentage are abnormally high.

Results of the LCCA shall be used as a tool to aid in pavement type selection and to select appropriate pavement design strategies. For rehabilitation of existing pavements, LCCA must be conducted where major rehabilitation (such as total reconstruction, rehabilitation, and so on) is necessary or where options of different life expectancies are being considered. Projects not requiring an LCCA under this section require a cost analysis to compare the construction costs for each alternative. The report should include a discussion of the cost analysis and justification for the chosen alternative.

This is to satisfy the Pavement Type Selection process early in project development and does not constitute an approved pavement design.

10.10.3 LCCA Methods

Two approaches to LCCA may be employed - deterministic and probabilistic. Traditional LCCA procedures utilize deterministic analysis procedures, such as input factors are expressed as single "fixed" values without regard to the variability of the input factors.

These procedures are appropriate when the input factor variables (such as unit costs or timing of rehabilitation) are reasonably well known. However, sensitivity of the results to the input variables should be checked by adjusting the input variables to the high and low end of their expected values, such as *best-case* and *worst-case* scenarios, recalculating the life cycle cost and re-evaluating the results.

Deterministic procedures are appropriate when one alternative appears to have a clear economic advantage over other alternatives under both best-case and worst-case scenarios. An example of this is when Alternative A has a lower life cycle cost than Alternative B even when the input variables are chosen to handicap Alternative A and favor Alternative B.

This concept of sensitivity can be taken one step further by performing a probabilistic LCCA. Probabilistic LCCA is a relatively new approach involving risk analysis and is considered good practice by FHWA. This process involves Monte Carlo simulation to incorporate variability of the LCCA inputs.

This technique is encouraged when there is a considerable amount of uncertainty in the input variables or when it is desirable to obtain a probability distribution of the results. This technique is also appropriate when the favored alternative in a deterministic analysis switches depending on the values used for the input variables.

The probabilistic approach to LCCA is documented in a FHWA September 1998 Interim Technical Bulletin entitled "Life Cycle Cost Analysis in Pavement Design -In Search of Better Investment Decisions." This document will be referred to hereinafter as the September 1998 FHWA Bulletin. Please refer to this manual for a detailed explanation of the procedure.

10.10.4 General Approach to LCCA

When an LCCA analysis is applicable, it should be conducted as early in the project development cycle as possible. The level of detail should be consistent with the level of investment. The general approach to a life cycle cost analysis is illustrated in the following steps:

a. Develop the new work or pavement rehabilitation alternatives to be considered.

b. Determine the length of the analysis period and the discount rate.

c. Determine the performance period and sequence of rehabilitation for each alternative over the duration of the analysis period.

d. Determine the agency cost for each alternative and rehabilitation strategy.

e. Evaluate user costs for each strategy (if appropriate).

f. Compute Net Present Value (NPV) for each alternative.

g. Review and analyze the results.

h. Adjust input variables and re-run the analysis to determine the sensitivity of the results to the input variables (best-case / worst-case scenarios).

i. Use the data to assist in selecting the appropriate alternative.

j. The September 1998 FHWA Bulletin includes a discussion of *constant* or *nominal* dollars to estimate future costs. The bulletin recommends that costs be estimated in constant dollars and discounted to the present using a real discount rate. This combination eliminates the need to estimate and include an inflation premium for both cost and discount rates.

According to the September 1998 FHWA Bulletin, Net Present Value (NPV) is the economic efficiency indicator of choice. The Equivalent Uniform Annual Cost (EUAC) indicator is also acceptable, but should be derived from the NPV. Both indicators should be calculated for GDOT projects. This will enable the decision-makers to compare the annual cost and see if maintenance costs could affect the results.

10.10.5 Analysis Period

According to the September 1998 FHWA Bulletin, the LCCA analysis period should be sufficiently long to reflect the long-term cost differences associated with the design strategies. As a rule of thumb, the analysis period shall be long enough to incorporate at least one rehabilitation activity for each alternative. Regardless of the analysis period chosen, the analysis period shall be the same for all alternatives. For new construction or projects with extensive pavement rehabilitation, a 30 and 40 year analysis period is appropriate. For projects where pavement design alternatives are developed to temporarily improve the pavement serviceability (for instance 10 years) until total reconstruction, a shorter analysis period is appropriate.

10.10.6 Discount Rates

Discount rates are used to convert future expenditures into equivalent current costs. Real discount rates reflect the true value of money with no inflation premium and should be used in conjunction with non-inflated cost estimates of future investments.

Because discount rates can significantly influence the analysis results, LCCA should use a reasonable discount rate that reflects historical trends over a long period of time. Higher discount rates typically favor lower initial costs and higher future costs. Lower discount rates do the opposite. The long term trend for real discount rates ranges from about 3 to 5 percent with an average of about 4 percent according to the September 1998 FHWA Bulletin.

10.10.7 Establishing Strategies, Performance Periods and Activity Timing

Feasible and reasonable strategies must be established for initial construction and subsequent maintenance and rehabilitation. These strategies must be developed using the pavement design guidelines described in other sections of this guide. Where applicable, designs must consider future modernization. Unrealistic or inappropriate strategies to favor one particular alternative shall not be used.

Information on performance for various pavement strategies may be obtained from Pavement Management System (PMS) data if available and from historical records or experience. The Designer may need to look at similar projects in the area to determine the expected life range for the analysis. If no other data is available, expert opinions should be gathered and documented as to the reasoning for the expected performance period for the rehabilitation type.

10.10.8 Cost Ranking of Alternatives

Following the completion of the LCCA analysis, GDOT ranks the alternatives using a multi-criteria analysis matrix. This matrix has weights assigned as a percentage to criteria or factors in the LCCA analysis, such as construction costs, maintenance costs, and user delay costs.

Decision Matrix factors can be further broken down as follows:

- Major Factors are LCCA criteria that have readily quantifiable costs (unit costs, more certainty in their values)
- Minor Factors which are LCCA criteria that have less readily quantifiable costs (less certainty in their values).

Examples of Major Factors or Criteria with readily quantifiable unit costs, available from historical bid prices listed in the Departments' *Mean Item Summary*, are:

- Material Costs
- Traffic Control Costs
- Construction Costs

Examples of Minor Factors or Criteria with less readily quantifiable unit costs are:

- User Delay Costs
- Familiarity with construction of proposed pavement type
- Performance of proposed pavement type on other projects
- Conservation / Recycling of Materials
- Stimulation of Competition between construction industries

10.11 Project Costs

10.11.1 Agency Costs

The LCCA need only consider differential costs between alternatives, which are typically the costs for the pavement components. Costs common to all alternatives will cancel out. These cost factors are generally noted and excluded from LCCA calculations. Additional cost items that may vary between alternatives such as temporary pavement for staging, and adjustment of structures, barriers, or guardrails shall be evaluated for each alternative.

10.11.2 Initial Project Construction and Rehabilitation Costs

Agency costs include all costs incurred directly by the agency over the life of the project. They typically are dominated by construction costs but also include initial preliminary engineering, contract administration, and construction supervision costs. Unit costs will typically be determined by the GDOT bid price data on projects with quantities of comparable scale and geographic location.

Information on rehabilitation cycles for various pavement types may be obtained from Pavement Management System (PMS) data if available and from historical records or experience. The Designer may need to look at similar projects in the area to determine the expected life range for the rehabilitation. If no other data is available, expert opinions should be gathered and documented as to the reasoning for the expected performance period for the rehabilitation type. The following cycles have been commonly used for GDOT LCCA for a given pavement type.

Pavement Type	Cycle
Asphalt	Every 10 years: 5% Deep patching, Mill & Inlay
JPCP	Every 20 years: Grind, 5 % Slab replacement, waterproofing joint and cracks
CRCP	Every 25 years 2.5% punch-out repair

10.11.3 Maintenance Costs

Routine, reactive type maintenance costs have only a marginal effect on NPV. These are hard to obtain, and are generally very small in comparison to initial and rehabilitation costs. Cost differences between maintenance strategies for two competing alternatives are usually small, especially when discounted over the analysis period. Therefore, maintenance costs will not normally be considered in the analysis.

10.11.4 Salvage Value

Salvage value represents the value of an investment alternative at the end of the analysis period. It is primarily used to account for differences in remaining pavement life between alternative pavement design strategies at the end of the analysis period. It will be based on the remaining life of the alternate at the end of the analysis period as a prorated share of the last rehabilitation cost. The salvage value is included as a negative cost.

For example: if a 35 year analysis is conducted and a $100,000 rehabilitation strategy with a 10-year design life is applied in year 30, the salvage value at year 35 is calculated by multiplying the percent of design life remaining at the end of the analysis period (5 of 10 years or 50 percent) by the cost of the rehabilitation ($100,000 in this example).

10.11.5 User Costs

This topic is referred to in detail in the September 1998 FHWA Technical Bulletin. User costs are the delay, vehicle operating, and crash costs incurred by users of the facility.

According to the September 1998 FHWA Bulletin, vehicle delay and crash costs are unlikely to vary among alternative pavement designs between periods of construction or maintenance. Although vehicle-operating costs may vary between pavement design strategies, there is little research on quantifying such cost differentials under the pavement condition levels prevailing in the USA.

When work zone capacity exceeds vehicle demand of the facility, differences in user costs between pavement design strategies are minimal and represent more of an inconvenience rather than a serious cost to the traveling public. This is the typical case for most GDOT projects.

User costs may become a significant factor when a large queue occurs on one alternative but not the others. For those projects in locations where one of the alternatives being considered will create a significant queue for an extended period of time either during initial construction or rehabilitation, a user cost analysis should be considered in addition to an agency cost LCCA.

Agency costs and user costs shall be evaluated separately. The results shall not be added together at the end to provide one cost for a given alternative.

10.12 Interpreting and Presenting Results

Once completed, the LCCA should be subjected to a sensitivity analysis to evaluate best-case and worst-case scenarios. The sensitivity analysis can be used to develop a feel for the impact of variability of the individual inputs on the overall LCCA results.

A common situation is to evaluate the LCCA for various discount rates. Variations in unit costs or activity timing can also have a significant effect on the NPV. Summary tables or plots of NPV versus individual input variables are useful in interpreting these results. This information must also be included in the pavement design report.

Where life cycle costs between alternatives exceed 10%, the pavement design alternative with the lowest life cycle cost will typically be the preferred alternative. However, when alternatives have comparable life cycle costs, other factors may be used to base a decision.

The final selection of an alternative is agreed to by the Project Manager and the Pavement Management Branch. The LCCA study may be presented to the Pavement Design Committee for informational purposes.

For final approval of an alternative, a consensus decision should be reached among the Pavement Design Committee Members.

In addition to LCCA, other issues shall be factored into the selection of a given alternative, including but not limited to:

- Initial Construction Agency Costs
- Maintenance Costs (nominal and discounted)
- Annualized Agency Costs
- Annualized User Costs
- Salvage Value
- Expected Life (Rehabilitation Frequency)
- Construction (production rate - initial days)
- Ease of Repairing / Maintaining (production rate - rehab days)
- Constructability / Traffic Control (Lifts)
- Proven Design in Agency

10.13 Pavement Type Selection Summary

The Pavement Type Selection process consists of the following separate and distinct, yet related parts.

10.13.1 Part I: Field Engineering and Design

- Complete a Pavement Evaluation if any existing pavement is being retained.
- Develop several pavement design alternates for comparison.
- Plan appropriate maintenance treatments at regular intervals for the various design alternates.

10.13.2 Part II: Economic Analysis

- Perform a LCCA comparing the different pavement designs proposed, including their maintenance.
- Incorporate user delay costs for all construction periods.
- Weigh-in the results of the LCCA comparing different pavement designs using a multi-criteria analysis matrix.

10.13.3 Part III: Engineering Judgment

- Incorporate the Project Manager's experience and common sense.
- Recommend the most suitable design alternate.

Thus a good Pavement Type Selection Process is the following:

- Establishes a method for selecting the preferred pavement alternate for the given project or corridor.
- Is part of a comprehensive Pavement Management approach.
- Takes into account the total construction and user delay costs over the life of the pavement (LCCA).
- Incorporates the designers' experience and recommends the most suitable design alternate
- Is a Project Specific Process.
- Is applicable to Major Projects.
- Must be justifiable to GDOT management (i.e. makes the entire process a transparent one by using a thorough and proven analysis methodology based on consistent, verifiable parameters)

Projects for which no PTS is needed are the following project types meeting the Minor Project Pavement Design Guidelines.

- Routine maintenance projects
- Safety Improvement projects such as intersection improvements
- Bridge replacement projects
- Passing lane additions

10.13.4 Pavement Type Selection in General

GDOT evaluates every project on a case by case basis. If a project is part of a corridor, then that is another factor that is taken into consideration as well when a pavement type selection is proposed, for the following type projects:

- GRIP Corridors
- Interstate Widening and Maintenance Rehabilitation

- Major Arterial Projects in Urban Areas
- Major Maintenance Reconstruction Projects
- New Corridor Widening / New Construction

In general projects should be based on the following guidelines:

- If the project is new construction, at least two pavement types may be considered and if an LCCA is deemed necessary, then one of the pavement types considered will be recommended.
- If the project is a total reconstruction project then the existing pavement is considered to have reached the end of its useful service life. For this type project, at least two pavement types may be considered and following an LCCA, one of the pavement types considered will be recommended.
- If the project is a rehabilitation project then the lanes are typically in fair to poor condition:
- If the existing pavement is flexible and it is in good condition then, a thin overlay is provided and it is based on traffic requirements.
- If the existing pavement is flexible, and it is in fair condition, then a mill and inlay is provided and it is based on traffic requirements.
- If the pavement is rigid, then selective slab or partial slab replacement, dowel bar retrofit, or other suitable rehabilitation technique will be recommended.

If the project is a widening project, then the recommended pavement type for the new addition will be chosen as follows:

- If the pavement on the existing lanes is in good condition, then the same pavement type is recommended for the additional lane(s).
- If the existing pavement is in fair condition, and if :
  - The Existing Pavement is Flexible: then partial or full depth reconstruction would be considered, and the following two options are considered:
    - Partial depth reconstruction may be accomplished with an AC pavement.
    - If the mill depth is such that a rigid pavement may serve just as well, then a rigid pavement (overlay) is also recommended.
    - Project constraints may favor one type over another.

- The Existing Pavement is Rigid:
    - A flexible overlay, to extend the service life, is considered only if the intention is to reconstruct in the future.
    - A rigid overlay is considered if there are no immediate plans for reconstruction.
- The Existing Pavement is Composite:

    The depth and condition of the existing asphaltic concrete over the PCC pavement is determined.

    - If the condition of the asphaltic concrete is poor, then the asphaltic concrete will be removed and 3 inches of 19 mm AC is placed on the existing PCC and a concrete overlay is placed on the AC.
    - If the condition of the asphaltic concrete is not poor, then we recommend milling and inlay with a depth of asphaltic concrete that is determined by traffic needs.
    - Additional lab and field studies are underway to establish a minimal depth of AC in composite sections that can support loads without being fatigued by high load stresses to failure.
    - If the existing pavement is in poor condition, and full depth reconstruction is deemed warranted, then a flexible and rigid pavement design alternates are developed, and compared in an LCCA. Final pavement type selection involves project manager input as well.

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11 Pavement Design

Pavement Design is the process of developing the most economical combination of pavement layers, with respect to thickness and type of material, to protect the soil foundation from the cumulative traffic loading that is anticipated to be carried during the design life.

The pavement design approach can be outlined in the following steps:

1. Quantify the loading conditions of this pavement by estimating the number of anticipated load repetitions that will occur over the design period.
2. Define the environmental conditions in which this pavement will be located.
3. Select economical, locally available construction materials with the appropriate engineering properties for use in the construction of this pavement.
4. Determine the thickness of the pavement based on empirical rules, or determine the thickness of the pavement based on a stress – strain analysis of the pavement structure; and under conditions and criteria, conforming to the Pavement Type Selection Process, revise the initial pavement structure design.

   Note: This revised pavement structure may simply be a similar pavement with a different combination of pavement layer thicknesses or may be a different pavement type altogether.

5. Repeat this process until a satisfactory pavement design is achieved. This design satisfies applied loading and environmental conditions, locally available construction and subgrade materials.

Although the design approach is relatively simple to express, the solution for a pavement structure is more involved and complex.

11.1 Pavement Structure Types

Pavements are divided into two broad categories: flexible pavements and rigid pavements.

Flexible Pavement Structures - Flexible pavements are so named because they flex under the actions of traffic and rebound when traffic loads are removed. They consist of a base material that has been overlaid by asphaltic concrete layers. Typical asphaltic concrete layers, specified for state routes, consist of a base asphaltic layer, a binder layer and a surface layer. The surface mix specified on higher volume, higher duty routes, may differ from typical state routes. Georgia Interstate routes, freeways and high volume arterials have an additional asphaltic concrete layer, in addition to the higher duty surface layer, that assists in draining water from its surface. This draining function improves safety and traction by reducing splash back during a storm event.

Rigid Pavement Structures - Rigid pavement structures on the other hand do not flex as much as flexible pavements under the actions of traffic loading. They resist applied loadings by slab action. Dowels, connecting adjacent slabs, assist in the load transfer, assure a monolithic behavior of adjacent slabs, and reduce the faulting. Widened slabs also assist in the load carrying capability, thereby reducing deflections and stresses at the extreme fibers of the concrete. Rigid pavement types, most commonly used by the Department include the following:

- Jointed Portland Cement Concrete Pavements (JPCP)
- Continuously Reinforced Concrete Pavements (CRCP)
- Their variants for overlays

11.2 Available Design Methods

At the time of its completion, the AASHO Road Test represented the most comprehensive development of the relationships between performance, structural thickness and traffic loadings of pavements. The results were limited by the scope of the test and the conditions under which they were conducted. Pavement design procedures that were based on the empirical results of the AASHO Road Test were supplemented by existing design practice and available theory. A summary of its findings are presented in Appendix B.

Over the years, the following Design Guides have evolved as a result of this road test and have been available for the design of pavement structures:

11.2.1 The 1972 AASHTO Interim Guide for Design of Pavement Structures

The design procedures outlined in the 1972 AASHTO Interim Guide make certain assumptions in applying the Road Test equations to mixed traffic conditions and to situations where the soil materials and climate differ from those that prevailed at the test site.

The performance equations from the Road Test were predicated on a specific set of paving materials and one subgrade; a single environment; an accelerated procedure for accumulating traffic and accumulating traffic on each test section by operating vehicles with identical loads and axle configurations, rather than by mixed traffic. This is the design guide that has been adopted by the Georgia DOT for the design of Pavement Structures.

11.2.2 The 1981 AASHTO Revision to the 1972 Interim Guide

In 1981, Chapter 3 of the 1972 Interim Guide, was revised. This Chapter was on the design of PCC Pavements. It featured more detailed nomographs for PCC design.

11.2.3 The 1986 & 1993 AASHTO Pavement Design Guides

In the 1986 and 1993 editions of the *AASHTO Pavement Design Guide*, the concepts of resilient modulus of the subgrade, and reliability in design were introduced, in addition to the previous parameters that were considered in the 1972 Interim Design Guide and its 1981 Revision of Chapter 3 on Rigid Pavements. Both flexible and rigid pavement design equations were revised to incorporate those new concepts.

11.2.4 The 1998 AASHTO Revision – Concrete Supplement

In 1998 AASHTO added a supplement specifically dealing with rigid pavement structure rehabilitation considerations.

11.2.5 NCHRP 1-37a Mechanistic-Empirical Design

Currently the GA DOT does not use the NCHRP 1-37a Mechanistic-Empirical Design procedures for the design of pavements. However, a brief description of the Mechanistic-Empirical Design objective, purpose and history are presented below for the reader's knowledge. If and when GA DOT changes to this design procedure, the Pavement Design Manual will be revised accordingly.

Objective - The overall objective of the Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (referred to hereinafter as the Design Guide) is to provide the highway community with a state-of-the practice tool for the design of new and rehabilitated pavement structures, based on mechanistic-empirical principles. This objective was accomplished through developing the following:

- The Design Guide itself, which is based on comprehensive pavement design procedures that use existing mechanistic-empirical technologies.
- User-oriented computational software and documentation based on the Design Guide procedure.

Purpose - The design Guide represents a major change in the way pavement design is performed. The designer first considers site conditions (traffic, climate, subgrade, existing pavement condition for rehabilitation) and construction conditions in proposing a trial design for a new pavement or rehabilitation. The trial design is then evaluated for adequacy through the prediction of key distresses and smoothness. If the design does not meet desired performance criteria, it is revised and the evaluation process repeated as necessary. Thus, the designer is fully involved in the design process and has the flexibility to consider different design features and materials for the prevailing site conditions.

This approach makes it possible to optimize the design and to more fully insure that specific distress types will not develop.

The mechanistic-empirical (M-E) format of the Design Guide provides a framework for future continuous improvement to keep up with changes in trucking, materials, construction, design concepts, computers, and so on. In addition, guidelines for implementation and staff training have been prepared to facilitate use of the new design procedure, as well as strategies to maximize acceptance by the transportation community.

History - The nation's highways reached an estimated 2.7 trillion vehicle-miles in 2000. This is four times the 1960 level. This amounts to 7.4 billion vehicle-miles of travel every day. Truck travel (single-unit and combinations) has increased 231 percent since 1970. Combination truck travel has increased 285 percent over 1970 levels and now accounts for 4.9 percent of total annual vehicle-miles of travel versus 3.2% in 1970. (1) The 4 million miles of U.S. roadways (with 2 million miles of paved roads) have been constructed, rehabilitated, and maintained over the previous century, and they represent a huge national investment that has provided a safe and comfortable means of transportation for both private and commercial vehicles. Highways have contributed significantly to the economic growth of the nation.

- Pavement structures wear down and deteriorate under heavy axle loadings and exposure to the elements (very hot and very cold temperatures, freezing and thawing, precipitation). Therefore, they must be maintained and improved on a regular basis. This requires a very significant commitment of resources on the part of the nation's highway agencies (State, Federal, and local). Figure 11.1 illustrates the magnitude and increasing level of highway expenditures by function from 1980 to 2000.

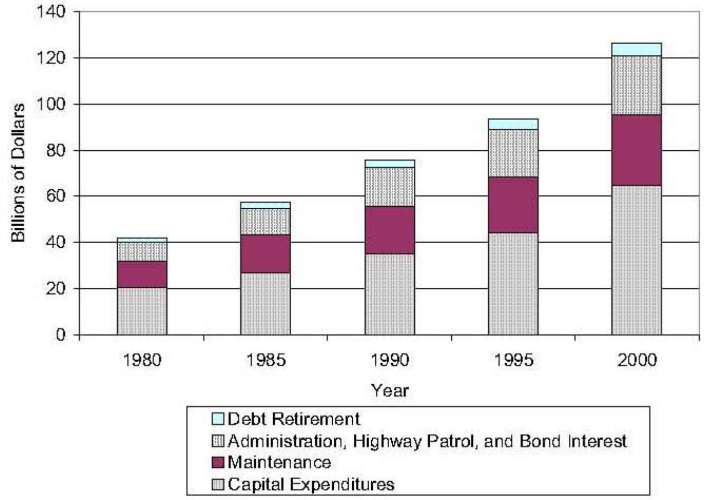


FIGURE 11.1 THE MAGNITUDE AND INCREASING LEVEL OF HIGHWAY EXPENDITURES BY FUNCTION FROM 1980 TO 2000.

Total highway expenditure by all units of government in 2000 was $126.7 billion, a 203 percent increase compared to 1980 (average annual increase of 10 percent). Note from Figure 11.1 that the annual level of expenditure is clearly accelerating over time. The 2000 total disbursement by State highway agencies was $89.8 billion, of which 53.1 percent went to capital outlays which includes 10.5 percent for new highway construction and 42.6 percent for improvements on existing highways. Approximately one-half of the capital outlay goes to pavement related work. The sheer magnitude of annual expenditures on highway pavements justifies the application of the best available design procedures to optimize the use of highway funds. Any improvements in design of new or rehabilitated pavement structures will have significant and sizeable implications in reducing the cost of maintaining these highway pavements.

As of the publication of this Design Guide, the *AASHO Guide for Design of Pavement Structures* is the primary document used to design new and rehabilitated highway pavements. The Federal Highway Administrations' 1995-1997 National Pavement Design Review found that some 80 percent of States use the 1972, 1986, or 1993 AASHTO Guides. All those versions are empirically based on performance equations developed using 1950's AASHO Road Test data. The 1986 and 1993 AASHTO Guides contain some refinements in materials input parameters, design reliability, and empirical procedures for rehabilitation design.

Since the AASHO Road Test, the AASHTO Joint Task Force on Pavements (JTFP) has been responsible for the development and implementation of pavement design technologies. This charge has led to many significant initiatives, including the development of every revision of AASHTO Guide. More recently, and in recognition of the limitations of the AASHTO Guide, the JTFP initiated an effort to develop an improved Design Guide. As part of this effort, a workshop was convened on March 24-26, 1996, in Irvine, California, to develop framework for improving the Guide. The workshop attendees (pavement experts from public and private agencies, industry, and academia) addressed the areas of traffic loading, foundation, materials characterization, pavement performance, and environment to help determine the technologies best suited for the new Design Guide. At the conclusion of that workshop, a major long-term goal identified by the JTFP was the development of a design guide based as fully as possible on mechanistic principles. This Design Guide is the end result of that goal.

11.3 The 1972 AASHTO Interim Guide for Design of Pavement Structures

Since the 1972 Design Guide has been adopted by GDOT, the following is a discussion of some the features it has considered. This also serves as a basis for future design guides, which have incorporated some changes and modifications to the design equations based on semi-empirical, semi-mechanistic considerations.

Georgia has developed lane-distribution factors for facilities with more than one lane in a given direction. These factors vary from 80 to 100 percent of the one-direction traffic for design of all lanes when there is a total of four lanes in both directions, and from 60 to 80 percent of the one-direction traffic to one or more of the outer lanes and lesser values to inner lanes when there are six lanes or more in both directions. If there is doubt, as to which factor to apply, it is suggested that the highest (most conservative) range be used.

Appendix A relates lane distribution factors to volumes and the number of lanes.

Included in the AASHTO design procedure is a regional factor (R), which provides an adjustment in the structural number for local environmental and other considerations. Suggested procedures by which the user agency may select appropriate regional factors are presented in the guide.

Factors considered in determining the regional factor: topography, similarity to Road Test location, rainfall, frost penetration, temperature, ground water table, subgrade type, engineering judgment, type of highway facility, and subsurface drainage. The Regional Factors for Georgia are included in Appendix H.

11.3.1 Flexible Pavement Structures

A flexible pavement structure may consist of at least three layers, subbase course, base course and surface course. The design procedure to satisfy the required structural number includes the determination of total thickness of the pavement structure, as well as the thickness of the individual components of the surface, base and subbase courses.

The structural number (SN) determined by this design procedure must be converted to actual thickness of surfacing, base and subbase layer by assigning as a layer coefficient to represent the relative strength of the material actually used for each layer. Layer coefficients are included in Appendix D.

The structural number is an abstract number expressing the structural strength of pavement required for a given combinations of soil support value, total equivalent 18-kip (80kN) single axle loads (ESAL), terminal serviceability index, and regional factor.

The subbase course is that portion of the flexible pavement structure between the sub grade and the base course. It usually consists of a compacted granular material. Either treated or untreated, or a layer of soil treated with a suitable admixture. In addition to its position in the pavement, it is usually distinguished from the base course material by less stringent specification requirements for strength, plasticity, and gradation as well as line and grade. Because it is obvious that the subbase course must be of significantly better quality than the roadbed soil, the subbase is often omitted if roadbed soils are of high quality.

In addition to the major function as a structural portion of the pavement, subbase courses may have additional secondary functions, such as: prevent intrusion of fine grained roadbed soils into the base courses, minimize the damaging effects of frost action, help in preventing the accumulation of free water within or below the pavement structure and provide a working platform for construction equipment.

The base course is the portion of the flexible pavement structure immediately beneath the surface course. It performs its major function as a structural portion of the pavement. It usually consists of aggregates such as crushed stone, crushed slag, crushed or uncrushed gravel or sand, or of combinations of these materials. It may be treated or untreated with stabilizing admixtures such as Portland cement, asphalt. or lime. Specifications for base courses are generally considerably more stringent than for subbase materials. For use in this design procedure, base material must be represented by a layer coefficient, in order that its actual thickness may be converted to a structural number.

The surface course of a flexible pavement structure consists of a mixture of mineral aggregates and bituminous materials, placed as the upper course and usually constructed on a base course. In addition to its major function as a structural portion of the pavement, it must also be designed to resist the abrasive forces of traffic, to reduce the amount of surface water penetrating the pavement, to provide skid-resistance, and to provide a smooth and uniform riding surface.

The solution of the design equation presented in this guide is in terms of a structural number (SN). The required SN must be converted to actual thickness of surfacing, base and subbase by means of appropriate layer coefficients representing the relative strength of material to be used for each layer. By solving the equation with the soil support value representative of the roadbed soil, an SN for the entire pavement is obtained and is represented by the general equation:

$$SN=a_1D_1 + a_2D_2 + a_3D_3 + \text{---} + a_nD_n \qquad \text{Equation 1}$$

where

$\mathbf{a_1, a_2, a_3, \ldots a_n}$ - layer coefficients representative of surface, base and subbase courses respectively, and

$\mathbf{D_1, D_2, D_3, \ldots D_n}$ - actual thickness in inches, of surface, base and subbase courses respectively.

The layer coefficient expresses the empirical relationship between SN and thickness, and is a measure of the relative ability of the material to function as a structural component of the pavement. Average values of layer coefficient for the materials used in the AASHO Road Test pavements were determined from the result of the test and were the following:

- Asphaltic concrete surface course = 0.44
- Crushed stone base course = 0.16
- Sandy gravel subbase course = 0.11

The design equation is presented in the form of two nomographs for simplicity of application. Separate nomographs are presented for a terminal serviceability index (P_t) of 2.5 (Figure 11.2) and 2.0 (Figure 11.3). Use a serviceability index of 2.5 for major highways, and 2.0 for other highways where a somewhat lesser level of serviceability may be tolerated. For design of temporary highways or stage construction that will not become part of the final pavement structure, use Pt of 2.0 and an appropriate traffic analysis period.

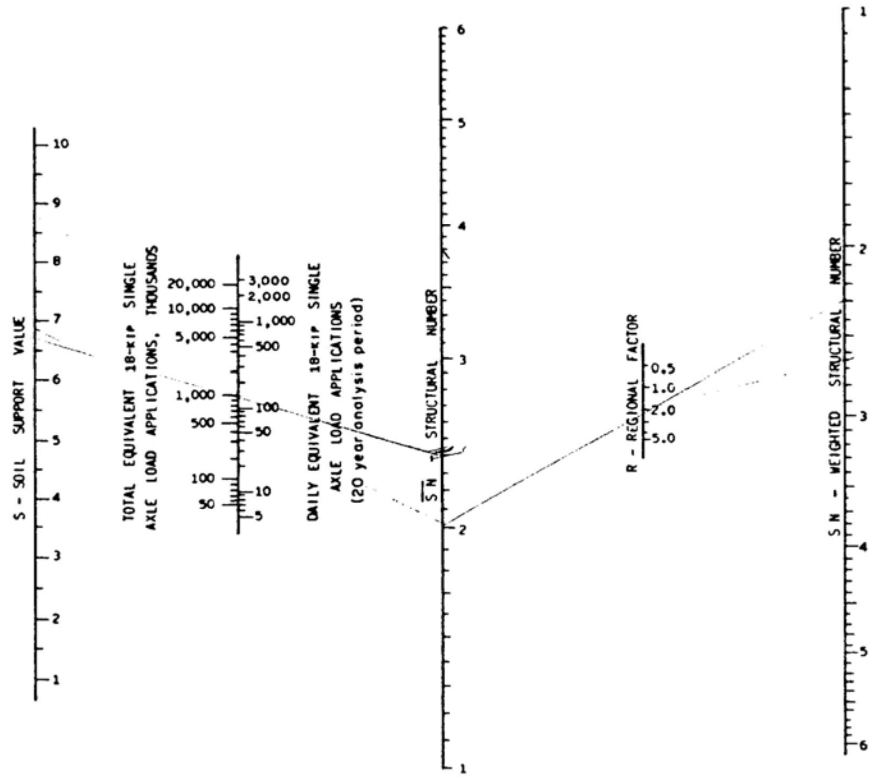


FIGURE 11.2 - FLEXIBLE PAVEMENT DESIGN NOMOGRAPH FOR A TERMINAL SERVICEABILITY OF 2.5

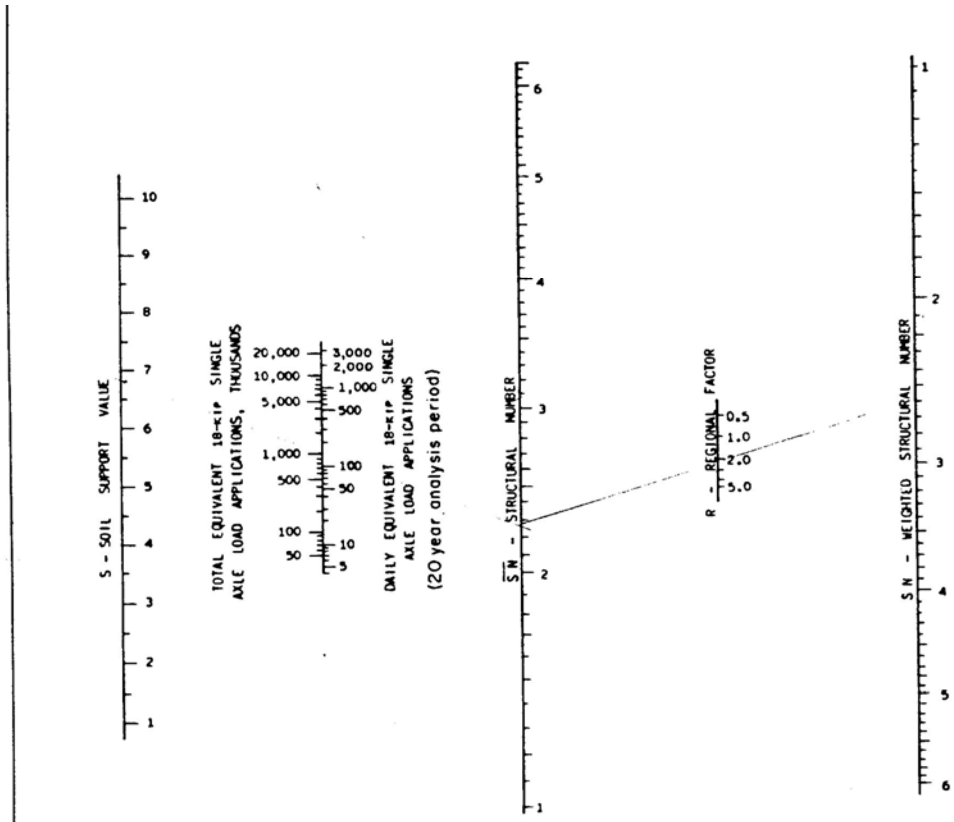


FIG 11.3 - FLEXIBLE PAVEMENT DESIGN NOMOGRAPH FOR A TERMINAL SERVICEABILITY OF 2.0

Once the decision has been made relative to the terminal serviceability index, select the appropriate design chart, determination should be made of the following:

- Representative values of Soil Support for the roadbed soil.
- The total or daily equivalent 18-kip (80kN) single-axle loads estimated for the design lane for the traffic analysis period.
- The regional factor is applicable to the site.

The chart requires two applications of a straightedge for each solution. First, the soil support value of the roadbed soil (on the left scale) and the total or daily 18-kip (80kN) single-axle loads for the traffic analysis period (left side of second scale) are used to solve for the unweighted structural number (center scale). This unweighted structural number is used with the selected regional factor (4^{th} scale) to solve for the design SN (right scale) applicable to the total pavement structure. Suitable designs are those whose combination of materials types and thicknesses satisfy the Equation 1.

11.3.2 Rigid Pavement Structures

Rigid pavement structures typically consist of at least four layers, designated as the pavement slab, the AC interlayer, the subbase, and the subgrade course. The design procedure includes the determination of the thickness of the Portland cement concrete pavement slab, and the design of the joints and of steel reinforcement. Also included are recommendations as to the treatment of subbase soils and the type and thickness of subbases required.

It is essential that the user of the design procedure in the guide understand its limitations, which are:

- The design chart scales for working stress (f_t) in concrete and modulus of subgrade reaction (k) are derived from the Spangler modifications of the Westergaard theory of stress distribution in rigid slabs.
- There is no adjustment in the AASHO Road Test rigid pavement equation for an environmental or regional factor.
- Although the traffic repetitions used in the development of the design relationship were experienced over only a two-year period, the traffic analysis period that must be selected for design is usually considerably longer than two years, typically 20 years. The traffic analysis period should not be confused with pavement life, which is affected by other factors in addition to traffic.

Two major overall assumptions have been made in the development of these design procedures, as follows:

- That the adequacy of the design will be established by soils and materials surveys and laboratory studies.
- That the design strengths assumed for the subgrade and pavement structure will be achieved through proper construction methods.

The subbase of a rigid pavement structure consists of one or more compacted layers of granular or stabilized material placed between the subgrade and the rigid slab for the following purposes:

- Provide uniform, stable, and permanent support.
- Increase the modulus of subgrade reaction (k).
- Minimize the damaging effects of frost action.
- Prevent pumping of fine-grained soils at joints, cracks and edges of rigid slabs.
- Reduce cracking and faulting.
- Provide a working platform for construction equipment.

The prevention of water accumulations on or in the subgrade soil or subbase is essential to attain satisfactory performance of the pavement structure.

The basic materials in the pavement slab are Portland cement concrete, reinforcing steel and joint sealing materials. Under the given conditions of a specific project, the minimum cement factor should be determined on the basis of laboratory tests and prior experience as to strength and durability. Air-entrained concrete should be used whenever it is found necessary to provide surface deterioration from freezing and thawing or from salt or to improve the workability of the mix. Other types of cement should be considered if materials in the area result in adverse reactions.

The reinforcing steel used in the slab should have deformations or deformation properties adequate to develop the working stresses in the steel. The steel mats required may be assembled on the project or prefabricated. There are numerous grades available for the requirements of the proposed use. In some cases, very high-strength steels are required, whereas in other cases the bending properties must be considered.

Two basic types of sealants are presently used for sealing joints:

- **Liquid sealants** - These include a wide variety of materials of three types: asphalt, hot-poured rubber and polymers. These materials are placed in the joint in a liquid form and allowed to set. When using liquid sealants, care should be taken to provide the proper shape factor for the movement expected.
- **Preformed Elastomeric seals** - These are extruded neoprene seals having internal webs that exert an outward force against the joint face. The size and installation width depend on the amount of movement expected at the joint.

The design procedure presented in this guide is a basis for the design of rigid pavement structures is based on data developed by the AASHO Road Test, supplemented and modified by theoretical analysis. It is in the form of nomographs (P_t = 2.0, P_t = 2.5) for ease in solution of the design equation.

Westergaard's modulus of subgrade reaction (k) (referred to as "gross k" in AASHO Road Test reports), is used in the guide. It represents the load in pounds per square inch on a loaded area divided by the deflection in inches of that loaded area. The scales for "k" included in the design charts are correlated with values obtained by plate loading tests performed in accordance with AASHTO DesignationT222 using a 30-inch (762mm) diameter plate. The "k" value may be estimated on the basis of previous experience or by correlation with other tests.

The scale included in the design charts is for working stress (f_t) in the concrete. A working stress of 0.75 times the modulus of rupture (S_c), as determined above, is recommended for use with these design charts. Although this working stress is 50 percent higher than that often used previously, it is considered satisfactory for two reasons:

The basic equation developed from the results of the AASHO Road Test is based on dynamic loadings, whereas previous design methods were based on a static load.

The design procedure in this guide is based on performance, rather than on the formation of the first crack.

The design charts were developed from equations based on protected corner conditions for jointed pavements, and the thickness of pavement slab determined from these charts applies to all jointed slabs, reinforced or non-reinforced, having provisions for adequate load transfer through mechanical devices or aggregate interlock.

GDOT designs all rigid pavements based on the 1981 revision to Chapter 3 of the1972 AASHTO Interim Guide for Design of Pavement Structures. Table 11.1 below lists rigid pavement design factors, and their typical values as used by GDOT.

Design factor	Symbol	Value
18k ESAL	MU SU Other	2.68 0.50 0.004
Initial Serviceability	Po	4.5
Terminal Serviceability	Pt	2.5
Serviceability Loss	ΔPSI	2.0
Modulus of Rupture	$S'c = 0.75 * f_r$	450 psi
Modulus of Elasticity of Concrete	$Ec = 57{,}000 * \sqrt{f'_c}$	3,200,000 psi
Poisson's Ratio of Concrete	ν	0.15
Modulus of Subgrade Reaction, k	k	Provided in Soil Survey Summary

TABLE 11.1 TYPICAL GDOT VALUES USED IN RIGID PAVEMENT THICKNESS DESIGN

In the design of continuously reinforced slabs, the slab thickness is identical to that of a Jointed Plain Concrete Pavement carrying the same traffic and under the same geotechnical and environmental conditions.

The following equation governs the design of rigid pavements according to the 1981 revision of the 1972 AASHTO Interim Guide for Design of Pavement Structures.

$$\log(ESAL) = 7.35*\log(D+1) - 0.06 + \frac{\log\left(\frac{4.5-P_t}{4.5-1.5}\right)}{1+\frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22-0.32*P_t)*\log\left[\left(\frac{f_r}{690}\right)\frac{D^{0.75}-1.132}{D^{0.75}-\frac{18.42}{\left(\frac{E_c}{k_{eff}}\right)^{0.25}}}\right]$$

Equation 2

where

ESAL - The total lifetime rigid ESALs anticipated to be applied on the pavement

D - The concrete slab thickness in inches

P_t - The pavement terminal serviceability index. GDOT's default value for rigid pavements is 2.5

f_r - Working stress of concrete = 0.75* S'$_c$= 0.75* 600 = 450 psi, a higher value can be used if properly documented, in accordance with the Standard Specifications for Construction of Transportation Systems, Section 430 or 439 and approved by the Concrete Branch at OMR.

E_c - Modulus of elasticity of concrete, the value used by GDOT is derived from the ACI Code.

E_c - Equals 3,200,000 psi for 3000 psi concrete. A higher value can be used if properly documented in accordance with the Standard Specifications for Construction of Transportation Systems, Section 430 or 439 and approved by the Concrete Branch at OMR.

k_{eff} - Effective modulus of reaction of subgrade and all underlying structural layers, such as GAB and bituminous bond breaking interlayer

Joint Types

The following are joint types for Rigid Pavement:

- Expansion Joint - The primary function of an expansion joint is to prevent the development of damaging compressive stresses due to volume changes in the pavement slab, and to prevent excessive pressures being transmitted to adjacent structures. In general it is considered that expansion joints are not necessary for rigid pavement, except adjacent to structures. At these locations expansion joints may be used when protected with satisfactory load transfer devices and suitable preformed joint fillers. A ¾- to 1-inch (19 to 25mm) width should be used. Where it is necessary to provide more than 1 inch (25mm) joints may be installed at intervals of approximately 20 feet (6m).
- Contraction Joint - The purpose of contraction joints is to provide for an orderly arrangement of the cracking that occurs. If the joints are properly designed and spaced, a minimum of cracking outside the joint would be expected. Contraction joints may be sawed in hardened concrete or formed by plastic inserts if performance indicates they are satisfactory. The depth of joint should be approximately ¼ of the thickness of the pavement slab. The design of the joint should be related to the expected joint opening, and the elongation of the joint filler used. Adequate load transfer through mechanical means or aggregate interlock should be provided at all joints. A 4- to 5-foot (1.2 to 1.5m) skew on a 24-foot (7.2m) width pavement will result in only one wheel crossing the joint at any one time. This skews results in better load transfer and improved riding quality across the joint. Random spaced joints have been used to prevent rhythmic or resonant reaction to a moving vehicle. Skewed joints have successfully been coupled with randomly spaced joints. GDOT has used skewed joints in the past and the current practice for joint layout and joint spacings are detailed in Ga. Std. 5046H.
- Longitudinal Joints - Are used to prevent the formation of irregular longitudinal cracks, and may be keyed butt joints or mechanically formed or sawed grooves. Adjacent lanes should be kept from separating and faulting by steel tie bars or connectors. The depth of formed or sawed grooves should not be less than ¼ the thickness of the pavement slab.

Mechanical load-transfer devices (dowels) should possess the following attributes:

- They should be simple in design, practical to install, and permit complete encasement by the concrete.
- They should properly distribute the load stresses without overstressing the concrete at its contact with the device.
- They should offer little restraint to longitudinal movement of the joint at any time.
- They should be mechanically stable under the wheel load weights and frequencies that will prevail.
- They should be resistant to corrosion when used in those geographical locations where corrosive elements are a problem.

Tie bars, either deformed steel bars or connectors, are designed to hold the faces of abutting slabs in firm contact. Tie bars are designed to withstand the maximum tensile forces required to overcome subgrade drag. They are not designed to act as load-transfer devices. Consideration should be given to the use of corrosion-resistant materials where salts are to be applied to the surface of the pavement.

- Shoulders - If the pavement has shoulders, it is preferable to have those shoulders as concrete shoulder and tied to the travel lanes. One major advantage of using tied concrete shoulders is to effectively increase the slab width resulting in a decrease in slab deflections. This results in the overall reduction of slab stresses, thereby increasing the service life they provide**.**
- Reinforcing Steel - The purpose of distributed steel reinforcement in reinforced concrete pavement is not to prevent cracking, but rather to hold tightly closed any cracks that may form, thus maintaining the pavement as an integral structural unit. The pavement slab tends to shorten when its temperature drops or its moisture content decreases. This contraction is resisted by the subgrade through friction and shear between it and the slab. The resistance to movement must be balanced by the tensile resistance of the steel crossing the crack. The maximum steel will occur at a crack at mid-length of a slab. Reinforcement is designed for the stress developed in this condition.

The percentage of longitudinal steel in a continuously reinforced pavement has been established by experience in experimental installations. It varies between 0.5 and 0.8 percent of the cross-sectional area of the pavement. Such variables as tensile strength of concrete, yield strength of steel, seasonal variations in temperature, and judgment based on experience should all be correlated in determining the percentage of steel.

Determination of the thickness of the pavement slab is accomplished by the use of the design charts in the following steps:

1. Select the applicable chart on the basis of the desired terminal serviceability index (P_t)
2. Using a straightedge, draw a line from the estimated total or daily 18-kip single axle loads on the left scale, through the applicable value of the working stress (f_t) of the concrete on the second scale, to intersect the pivot line.
3. With a second application of the straightedge, draw a line from the intersection of the pivot line to the applicable value of the modulus of subgrade reaction (k) on the right scale. The intersection of this line with the second scale from the right is the thickness of the pavement slab (D) in inches.

References:

Australian Stabilisation Industry Association, PO Box 797 Artarmon NSW 1570
1972 AASHTO Interim Guide for Design of Pavement Structures, AASHTO, Washington D.C.

11.4 Flexible Pavement Design

11.4.1 Flexible Pavement Design using WIN_APD

The current GDOT practice is to use the 1972 AASHO Interim Guide to design Asphalt Pavements. The Guide included nomographs and equations. The equations were rather complex and most designers utilized the nomographs to design their pavements. The nomographs used a log scale and depending on how well you understood log scales, the structural number that you determined from the nomographs varied from user to user. The Pavement Design Committee had questioned the resulting structural number as it tended to vary for similar conditions. One of our engineers took it upon himself to computerize the 1972 Interim Guide to help eliminate or minimize the discrepancies in the Structural Numbers. This computerized version is known as WIN_ APD, Asphalt Pavement Design (APD). The most current version is dated December 2001, and can be found at the OCD website.

The APD program is the Department's current design procedure for designing flexible pavements. The Department will transition into the NCHRP 1-37a (Mechanistic-Empirical Design) in the future.

CAUTION: If you QUIT or EXIT the program, any data that you entered is gone and you get to begin from nothing. Some minor glitches have been noted in WIN _ APD and it is recommended that if you have gone in and edited the program repeatedly, then once you have what you believe is an acceptable design, quit and restart the program, enter all your data and check your result against your first run.

There is some basic information you will need to get started, the AADT traffic diagrams, project length, a general description of the work, i.e., 4 lane rural section widening or new location, soil support value (SSV) and regional factor.

The traffic diagrams include the 24 hour truck percent as well as the percent Multi-unit (MU) and Single-unit (SU) trucks. The data we get in our traffic diagrams lump all multi-unit trucks together, any truck with a tractor and trailer is considered a multi-unit. All dump trucks, delivery trucks and busses are considered single-unit trucks. Pick-up trucks are not considered trucks but fall under the classification of automobiles. In the program only consider MU and SU trucks as their loads have a significant impact on the design of pavements. The program asks for the initial design traffic (one way AADT) and year and the final design traffic and year. It solves for the average traffic and looks at the difference in years as a whole number. The typical design period is 20 years. The program asks for the 24 hour truck percent; add this as a whole number.

The APD program is self explanatory. Open it up and follow the screens.

- The first screen contains the following basic project information: project number, project identification number, county, description, initial design year and design volume, final design year and design volume, truck percentage
- The second screen is used to provide more detailed information and contains a number of help screens or pull down menus.

The Lane Distribution Factor (LDF) is used to determine the amount of 18 kip ESALs in the design lane, typically as the number of lanes increase the LDF goes down. Currently the help screen goes up to a six lane section, three lanes each way. There is another source for determining the LDF. See *Appendix A* for a table that relates LDF to volume and the number of lanes. The LDF is added as a whole number.

Note: If you have a 4-lane urban, maybe a 4-lane connected to an interchange, or even a 4-lane rural roadway that is either heavy residential or commercial, then the trucks may be using all lanes almost equally either making left turns, avoiding right turning or entering vehicles. With a proliferation of driveways, side roads, and median openings to commercial areas the trucks may very well be using all lanes. Even the 4-lane ultimately tapering down into a 2-lane may be a consideration in your LDF factor. In these cases it could be the lower range.

The Terminal Serviceability Index (also known as Pt) is a rating of the pavement at the end of the design period, typically 20 years. During the Road Test, the pavements were rated at the initial opening to traffic and then rated at the end of the test, typically about a year's worth of traffic. The initial rating was set at 5.0 for new pavement that had a smooth ride. At the end of the test period it was rated again. This used pavement was still smooth riding, may have had some surface distresses and with some maintenance, (resurfacing) was still serviceable and rated at 2.5. If the pavement was worn out, had major distresses and needed reconstruction to be serviceable, then it was rated at 2.0. The program defaults to 2.5. If you are designing a detour paving that will not be incorporated into the final roadway then you should lower the Terminal Serviceable Index to 2.0, otherwise let it default.

The soil report contains the Soil Support Value (SSV) and a Regional Factor (RF). The Office of Materials and Research provides this information. Depending on where you are in the PDP (Plan Development Process) a soil report may not be available. The program has a help screen for your use in this case. If you do not have a soil report, you should note this in the remarks field so anyone looking at this design knows where the data came from. Additionally if this is a preliminary design say for concept, note that as well in the remarks.

The truck axle load factors have a default shown. Unless you have specific information regarding 18 kip ESALs use the defaults.

Representative 18 kip ESAL. Use the help screen. Your traffic diagrams show a 24 hour truck percent and a break down of the percent SU and MU trucks. For example, there are 16% trucks (24 hour) at 5% MU and 11% SU. Determine your 18 kip ESAL based on the ratio of MU vs. SU.

- 5%/16% = 31.25% MU
- 11%/16% = 68.75% SU

Using the help screen choose 30% MU and 70% SU for an 18 kip equivalent of 1.06.

The following screen lets you choose options for designing the pavements, Full Depth, Overlay or Editing. APD is set up to input thickness inches and in the output will list the thickness inches with millimeters (mm) in parentheses.

- Full Depth Design

The screen appears with pull down menus for the various courses. The surface course is usually a 9.5 mm Super pave at 1.25 inches (32mm) or 12.5 mm Super pave at 1.5 inches (38mm) for most projects. Projects with volumes less than 2000 ATD would use 4.75 mm Superpave at 7/8 inch (22mm), this would most likely be used with maintenance and LARP resurfacings.

If you are designing mainline pavement for an Interstate, use PEM (Porous European Mix) as the actual riding surface over a 1.5 inch (38mm) of 12.5mm SMA (Stone Matrix Asphalt). A State Route with current volumes of 25,000 two-way ADT and posted 55 MPH or greater will require a 12.5mm OGFC (Open Graded Friction Course). These PEM / OGFC courses do not add any structural value into your design.

The next pavement layer has been commonly referred to as the binder or intermediate course. 19mm Superpave at 2 inches (50mm) is the normal thickness. The maximum lift thickness is 3 inches (75mm) except for trench widening, typically 2 feet or less, where 4 inches (100mm) or 6 inches (150mm) can be specified.

Below the binder comes the asphalt concrete base. 25mm Superpave varying in depth from 3 inches (minimum lift thickness) (75mm) to 5 inches (maximum) (125mm) in 1 inch (25mm) increments. Trench widening can be 6 inches (150mm) thick.

There are three typical base materials, Graded Aggregate Base (GAB), Soil Cement Base and Superpave Base (Full Depth AC Base). GAB typically 8 inches (200mm) to 12 inches (300mm) in normally 2 inch (50mm) increments. Soil Cement can be an alternate base below the fall line, depending on availability of suitable soils. Check with OMR regarding the availability of suitable soils for a Soil Cement Base Alternate. The minimum thickness of Soil Cement Base is 6 inches (150mm) but the typical depth is 8 inches (200mm), with at least 6 inches (150mm) of Superpave over the Soil Cement Base. Construction requirements of Soil Cement Base usually limits it's use to new location projects or those projects with limited driveways since the Soil Cement Base has restrictions on allowing traffic across it when placed. When designing your base course, keep in mind the structural coefficients of the various materials, GAB 0.16 / inch (0.0063 / mm), Soil Cement 0.2 / inch (0.0079 / mm) and 25 mm Superpave 0.3 / inch (0.0118 / mm).

Note: The Interim Guide determined a required Structural Number (Sn or SN) and pavements are designed to the required Sn within a specific percent of under design depending on the shoulder type. Projects with rural shoulders shall be under designed between 10 and 15 percent. Projects with urban shoulders, curb and gutter, shall be designed between 0 and 5 percent under designed. The pavements are typically designed for a 20 year period and the Department typically resurfaces roads about every 10 years, therefore in 20 years there would be two resurfacings. On rural roads, two resurfacings would overlay the roadway 3 to 4 inches (75 mm to 100mm); and usually bring the roadway structural value up to the required Sn. On roadways with curb and gutter, the surface does not need to be raised 3 to 4 inches (75mm to 100mm), since this would introduce a drop-off at the gutter or reduce the height of curb. Designing the curb and gutter sections to the required Sn will necessitate future resurfacings to be a mill and inlay operation which is desirable, versus a simple overlay.

Once you have completed filling out the course menu, the program will compute and display the design. Does your design fit the conditions shown above? If yes, accept it and print, if not determine what needs changing, more or less Superpave or GAB and edit. The edit brings you back to the course menu. If you realize you want to change some of the input values, then accept it and then you move to the next screen which is the Pavement Design Form. At this point you can Quit, which exits you out of the program or go to Main Menu, giving you choices. Design Full Depth Pavement, Design Asphalt Overlay, Edit Design/Designers Data, or Quit.

- Overlay Design

The program uses the same data you stored for designing an overlay. You need to know the makeup of your existing pavement. Is it asphalt over what kind of base material? Your options include GAB, select material, or maybe concrete pavement. If you know this type of information then you can design an overlay. The designer should request a pavement coring and pavement evaluation. The core will tell you the depth of asphalt pavement and something about the base material, if it is GAB then you may have a depth as well. The pavement evaluation will provide more information and an assessment of pavement distresses, recommendations to correct these distresses, milling depths if necessary and overlay recommendations.

Note: Consider the position of the new pavement in respect to the existing. For example, the entire project will widen symmetrically about the centerline of the roadway with a 20 or 24 foot wide median or possibly a Two Way Left Turn Lane (TWLTL) with 4 through lanes. Should you be concerned with an overlay or even a pavement evaluation? Overlay, probably not, pavement evaluation probably so. Where are you in the PDP, Concept or Preliminary Plans? Pavement Evaluations take time to complete and provide good information. If the project follows the existing alignment where is the design lane? It is adjacent to the existing pavement, but it is not the existing pavement. The design lane carries 80 to 90 percent of the design loads, therefore the inside lane and median can carry, at the most 10 to 20 percent of the design loads. Therefore you need to edit the LDF to 10 to 20 percent. This reduces the 18 kip ESALs and reduces the Sn. The designer should rely on the recommendation from OMR for the Overlay.

The Overlay program is useful, but there are limitations and should be used cautiously. The design screen is broken down into two portions, one for the overlay and one for existing pavement information. It is set up like the Full Depth program. One note, the cores give you an average depth of say Asphalt, this is the only place where you should be exact as possible, no rounding to the nearest inch (25 mm). Do not consider leveling into the overlay design as this depth varies.

11.5 Rigid Pavement Design

Portland Cement Concrete (PCC) pavements are commonly referred to as rigid pavements. Rigid pavements respond to a wheel load as a very stiff material (concrete) over softer materials (subbase and subgrade).

The concrete layer in the rigid pavement develops bending moments from loading and acts as a slab to spread the wheel load over a large area of the subbase and subgrade, thus reducing stresses and strains in subbase and subgrade.

Typically there are three layers comprising a concrete pavement, a GAB subbase, an asphalt concrete base and the PCCP. Those are illustrated in the figure below which was obtained from the American Concrete Pavement Association (ACPA) website.

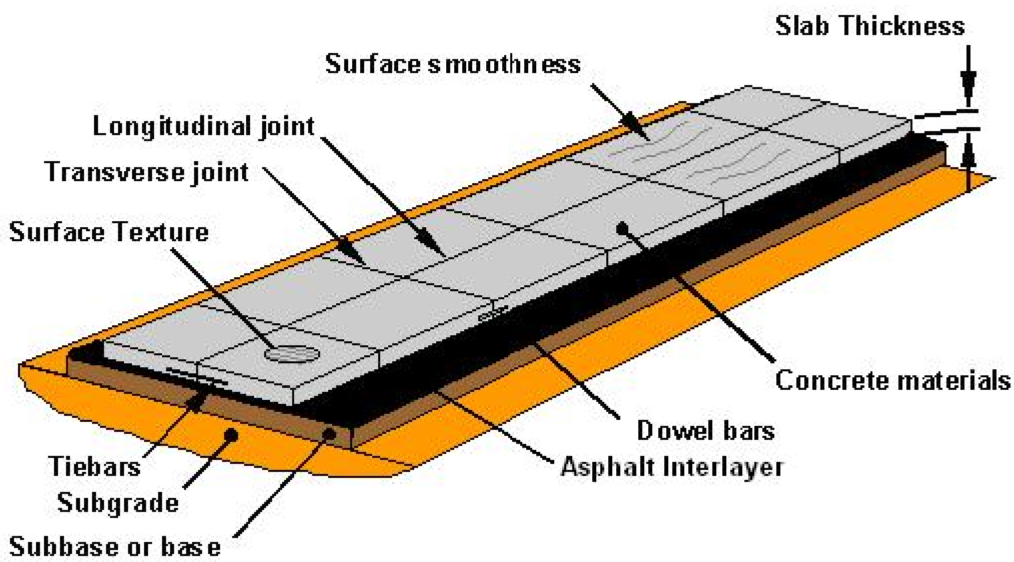


FIGURE 11.4 TYPICAL PCC PAVEMENT [3]

There are two types of rigid Portland Cement Concrete Pavements that are commonly used on Georgia D.O.T. projects.

- Jointed Plain Concrete Pavement (JPCP) - Jointed plain concrete pavement has transverse joints spaced at regular intervals. The transverse joints are used to control temperature induced contraction and expansion in the concrete. Smooth dowels are used at the transverse joints for load transfer. The transverse joints are spaced as shown in Ga. Std. 5046H. Longitudinal joints are used to control random longitudinal cracking. Longitudinal joints are tied together with tie-bars. Design details are governed by the following Design standards that can be obtained from GDOT's Internet web site at the following address: http://www.dot.state.ga.us/dot/preconstruction/roaddesign/downloads.shtml

- Continuously Reinforced Concrete Pavement (CRCP) - CRCP contains both longitudinal and transverse steel. CRCP does not contain transverse joints except at construction joints. The function of the longitudinal steel is not to strengthen the concrete slab, but to control concrete volume changes due to temperature and moisture variations and to keep transverse cracks tightly closed. The function of the transverse steel is to keep longitudinal joints and cracks closed. If the steel serves its proper function and keeps cracks from widening, aggregate interlock is preserved and concrete stresses in the concrete slab due to traffic loading are reduced. Steel reinforcement and other design details are governed by the CRCP Design standards that can be obtained from the GDOT's Internet web site at the following address: http://www.dot.state.ga.us/dot/preconstruction/roaddesign/downloads.shtml

Approved Design Method: The 1981 revision of Chapter 3 in the 1972 AASHTO Interim Guide for Design of Pavement Structures, (AASHTO Guide for Design of Pavement Structures – 1981 revision, American Association of State Highway and Transportation Officials. Washington, D.C. 2001) is the only approved design method for rigid pavements for GDOT.

11.5.1 Rigid Pavement Design Process

Thus far, all rigid pavement designs that are to be used on GDOT projects have been prepared by the Office of Materials and Research. In order to developing those designs, the Project Manager or the Design Consultant should gather and provide the following information for their submittals. Refer to the Rigid Pavement Design Analysis form in Appendix J.

Project Identification - designer provides the project identification number (P.I.N.), project number, county, project length, type selection (description) and project description. Noting the type of adjoining pavement is also required.

Traffic Data and Composition - The Office of Environment and Location (OEL) provides the traffic diagrams containing current and 20 year traffic projections, which include the 24 hour truck percent. The 24 hour truck percent is further broken down to show the multi-unit trucks (MU) and single unit trucks (SU).

Calculating Design ESALs

Do the following to calculate traffic data:

1. Add the 24 hour truck percentage.
2. List the one way AADT (initial year) and one way AADT (design year) to determine the mean AADT (one way).
3. The design loads are then calculated. Multiply the Mean AADT times the appropriate Lane Distribution Factor. This gives the mean traffic volume in the design lane.
4. Multiply the mean traffic volume in the design lane by the percentages of MU, SU and Others to obtain their mean design volumes.
5. Multiply the respective mean traffic volumes by the appropriate ESAL factors for MU, SU, and others to calculate the total mean daily 18 Kip equivalent axle loadings. The ESAL factors are shown in Table 11-2.
6. The total daily loads are multiplied by the number of days / year times the number of years for the analysis period to obtain the lifetime equivalent 18 kip axle loadings. Those are the loadings that are designed for.

Design factor	Symbol	Value
18k ESAL	MU	2.68
	SU	0.50
	Other	0.004

TABLE 11.2 18K ESALS-RIGID PAVEMENT

k-Value

The Office of Materials and Research (OMR) either provides or approves the k-value in the soil survey summary.

Rigid Pavement Design Preparation

The OMR Pavement Management Branch, Pavement Design Unit, prepares or reviews the rigid pavement design. This design is then presented to the Pavement Design Committee for review, discussion and approval.

Recommended Input Design Values

28-day Concrete Modulus of Rupture, f_r[1]

The f_r of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. An f_r of 600 psi at 28 days should be used with the current statewide specification for concrete pavement design. If the Engineer selects an alternate value to use for f_r, then it must be documented with an explanation, in accordance with the Standard Specifications for Construction of Transportation Systems, Section 430 or 439 and approved by the Concrete Branch at OMR.

28-day Concrete Elastic Modulus[1]

The Elastic modulus of concrete is an indication of concrete stiffness. It varies depending on the coarse aggregate type used in the concrete. Although the value selected for pavement design could be different from the actual values, the elastic modulus does not have a significant effect on the computed slab thickness.

A modulus of 3,200,000 psi should be used for pavement design. The use of a different value must be documented with an explanation, in accordance with the Standard Specifications for Construction of Transportation Systems, Section 430 or 439 and approved by the Concrete Branch at OMR.

Serviceability Indices[1]

For concrete pavement design, the difference between the initial and terminal serviceability is an important factor. An initial serviceability value, Po, of 4.5 and a terminal serviceability value, Pt, of 2.5 are to be used in the procedure, which results in a difference of 2.0.

Effective Modulus of Subgrade Reaction, k[1]

The slab support is characterized by the modulus of subgrade / subbase reaction, otherwise known as the k-value. It can be measured in the field by applying a load equal to 10 psi on the subgrade / subbase combination using a 30-inch diameter steel plate. The k-value is then calculated by dividing 10 psi by the measured deflection (in inches) of the layers under the plate. This k-value is provided in the approved Soil Survey Summary.

The k-value used for slab design is the effective k-value, k_{eff}, of all structural layers under the slab, typically GAB and an Asphalt Concrete interlayer. (see Tables 11.4 through 11.5)

Typical soil design values reported for GDOT Soil Survey Summaries are listed in the following table.

CBR	S.S.V.	Subgrade k-Value, pci	Typical GAB Thickness (inches)
3.4	2.0	110	12
4.1-4.2	2.5	130	12
5.3-5.5	3.0	150	10
6.7-6.9	3.5	175	10
9.0-9.5	4.0	190	8
11.3-11.9	4.5	200	8

TABLE 11.3 TYPICAL SOIL DESIGN VALUES IN GEORGIA

Instead of using a nomograph to determine the effective subgrade reaction modulus, k_{eff}, the following two tables can be used instead of the nomograph for that determination.

Step 1: Determine Effective k value over GAB Layer

Locate the appropriate GAB Layer thickness (12 inches), then the reported k-value in the soil survey summary (150). This gives an effective k value over the GAB layer of 245 pci. Table 11.3c below gives the effective k value at the top of a Graded Aggregate Base Course, the minimum thickness of which is specified in the Soil Survey Summary.

GAB Layer Thickness, inches >>>		8	10	12	14
k on top of Subgrade, k_{subg}, pci	Effective k over GAB Layer, k_{GAB}, pci				
100		145	165	195	220
110		155	175	205	230
120		165	185	215	240
130		175	195	225	250
140		185	205	235	260
150		195	215	**245**	270
160		205	225	255	280
170		215	235	265	290
180		225	245	275	300
190		235	255	285	310
200		245	265	295	320

TABLE 11.4 EFFECTIVE K OVER THE GRADED AGGREGATE BASE LAYER WITH A KNOWN K VALUE FOR THE SUBGRADE USING TYPICAL GEORGIA SOIL VALUES

GDOT's current practice is to use a 3 inch Asphalt Concrete interlayer placed between the slab and the Graded Aggregate Base course. Table 11.3d below lists effective k-values, k_{eff}, after a 3 inch AC interlayer has been added over the Graded Aggregate Base Course. This is the effective k-value that is to be used for in the Rigid Pavement design equation.

Step 2: Determine Effective k value over 3 inch AC Interlayer Layer

Locate the appropriate GAB Layer thickness (12 inches), then the reported k-value in the soil survey summary (150). This gives an effective k value over the 3 in AC Interlayer of 305 pci. This is the composite / effective k-value (k_{eff}) that represents the combined structural benefit of the subgrade, GAB, and AC Interlayer. This value will be used in the design equation or the nomograph used.

GAB Layer Thickness, inches >>>	6	8	10	12	14
k on top of Subgrade, k_{subg}, pci	Effective k over a 3-inch Asphalt Interlayer, k_{eff}, pci				
100	169	190	210	228	242
110	179	200	219	238	252
120	190	210	230	248	263
130	200	219	239	257	273
140	210	230	249	267	283
150	220	239	259	**305**	294
160	230	249	268	287	304
170	240	260	279	297	315
180	250	269	288	308	325
190	261	280	299	317	336
200	271	290	308	326	346

TABLE 11.5 CONVERSION FOR NOMOGRAPH, FIGURE 11.5

In summary for the above highlighted values:

k of subgrade	150 pci
k of subgrade and 12 inches GAB layer	245 pci
k of subgrade, 12 inches GAB layer, and 3 inch AC Interlayer	305 pci

Thus the 277 pci k-value combines the contribution of all structural layers.

Developing a Rigid Pavement Design Using Nomographs

The following steps are used to develop a rigid pavement design:

- Currently GDOT uses two nomographs to complete the design. The first nomograph (11.5) is used to determine k-value at the top of the subbase utilizing the k-value of the subgrade. This nomograph, Figure 11.5, has been converted from the 1972 AASHTO Interim Guide to provide additional clarity for the designer into Tables 11.4 and 11.5.
- Using Figure 11.5, enter your initial subbase thickness on the left, draw a horizontal line to the right until you intersect the curve representing the type of subbase you are proposing, typically GAB, turn and project the line vertically until you intercept the subgrade k-value and then turn back to the left and read the k-value on top of the subbase. Then enter the thickness of the asphalt base on the left extend the line to the right to intersect the curve for the asphalt base, turn and extend the line vertically until you intersect the k-value from the subbase, turn horizontally to read the k-value at the top of the base material. Tables 11.4 and 11.5 should be used to determine the k-value on top of the GAB and on top of the 3 inch interlayer. Using these tables should provide the designers with consistent k-values.
- Continue the solution by using the second nomograph. This nomograph solves the Rigid Pavement equation, Equation 11.2.

Note: If the AASHO Road Test equation is reduced for a Pt = 2.5, Equation 3 is the final result.

$$\log(ESAL) = 7.35 * \log(D+1) - 0.06 - \frac{0.17609125}{1+\frac{1.62*10^{7}}{(D+1)^{8.46}}} + 3.42 * \log\left[\left(\frac{f_t}{690}\right)\frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{\left(\frac{E_c}{k_{eff}}\right)^{0.25}}}\right]$$

Equation 3

Use the following process to solve Equation 3:

1. Using the second nomograph, (Figure 11.6) determine a slab thickness/actual stress in the concrete. Reading this chart from left to right you will see the load applications, actual stress in concrete, a pivot line, slab thickness in inches and k-modulus of subgrade reaction.

2. Follow along with the following example. Design a concrete pavement that can carry 20,000,000 loads on a subgrade k-value of 150 pci utilizing a 12 inch GAB subbase and a 3 inch asphalt base. Using the procedure described above you determine a k-value at the top of the subbase as 265 pci. Working through 3 inches of asphalt base (19mm Superpave) you determine that the k-value at the top of the asphalt is 277 pci.

3. Going to the second nomograph enter the k-value of 277 pci on the right most scale. Try a 10 inch slab and extend a line from the right to the left from k-value of 277 pci through the 10 inch trial slab thickness up to the pivot line. On the left side locate 20,000,000 loads and extend the line up to the pivot line intersection. This line crosses the actual stress in concrete at 540 psi.

   540 psi is greater than 450 psi so the pavement is over-stressed, or under-designed. Try a thicker slab; increase the thickness in one inch increments.

   An 11 inch slab gives you 450 psi as the actual stress in concrete; therefore, his is an acceptable design.

   An 11 inch PCCP over 3 inches of 19mm Superpave and 12 inch GAB is a solution to carry 20,000,000 18k ESALs over 20 years.

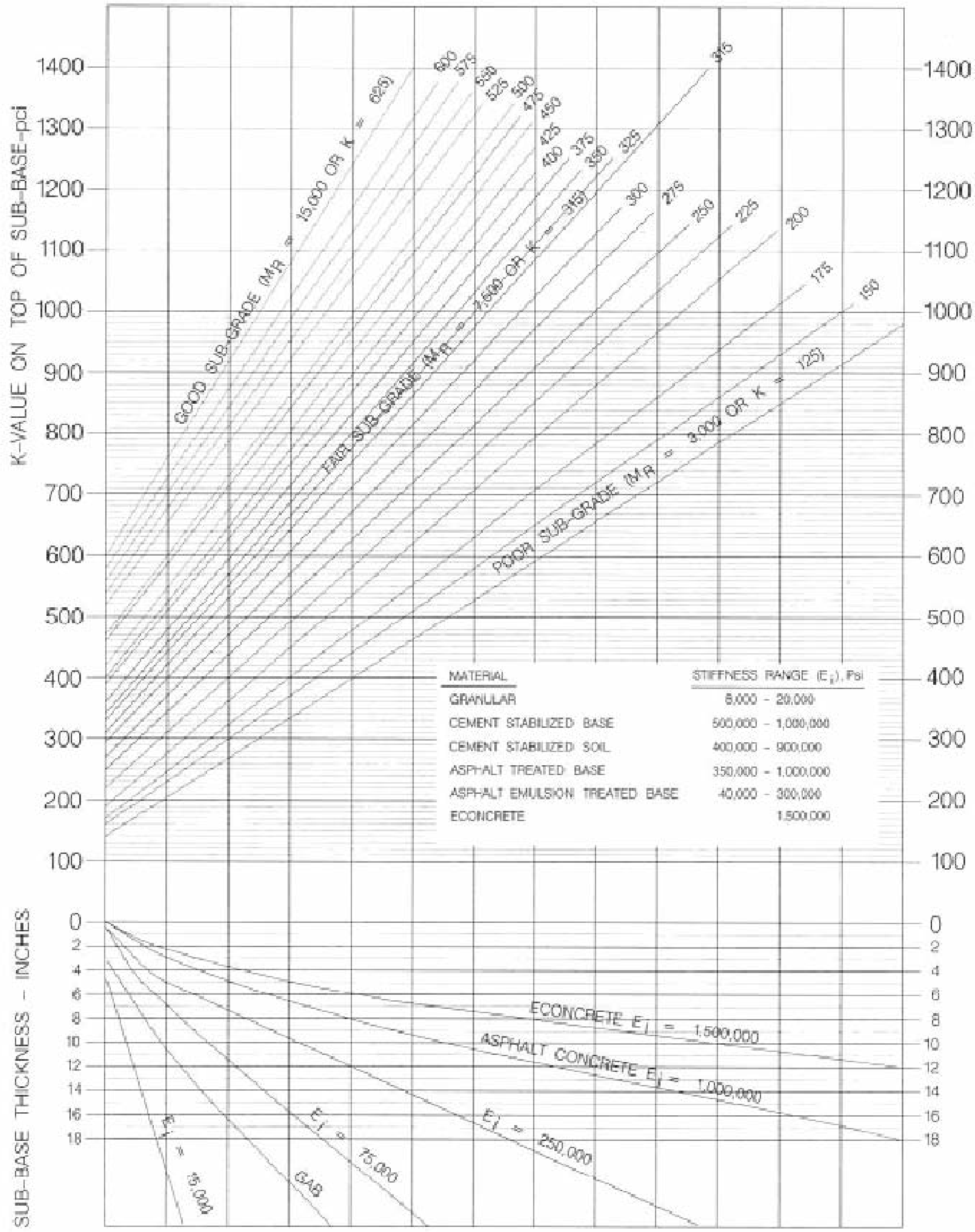


FIGURE 11.5 CHART FOR ESTIMATING SUB-BASE DESIGN K - VALUE

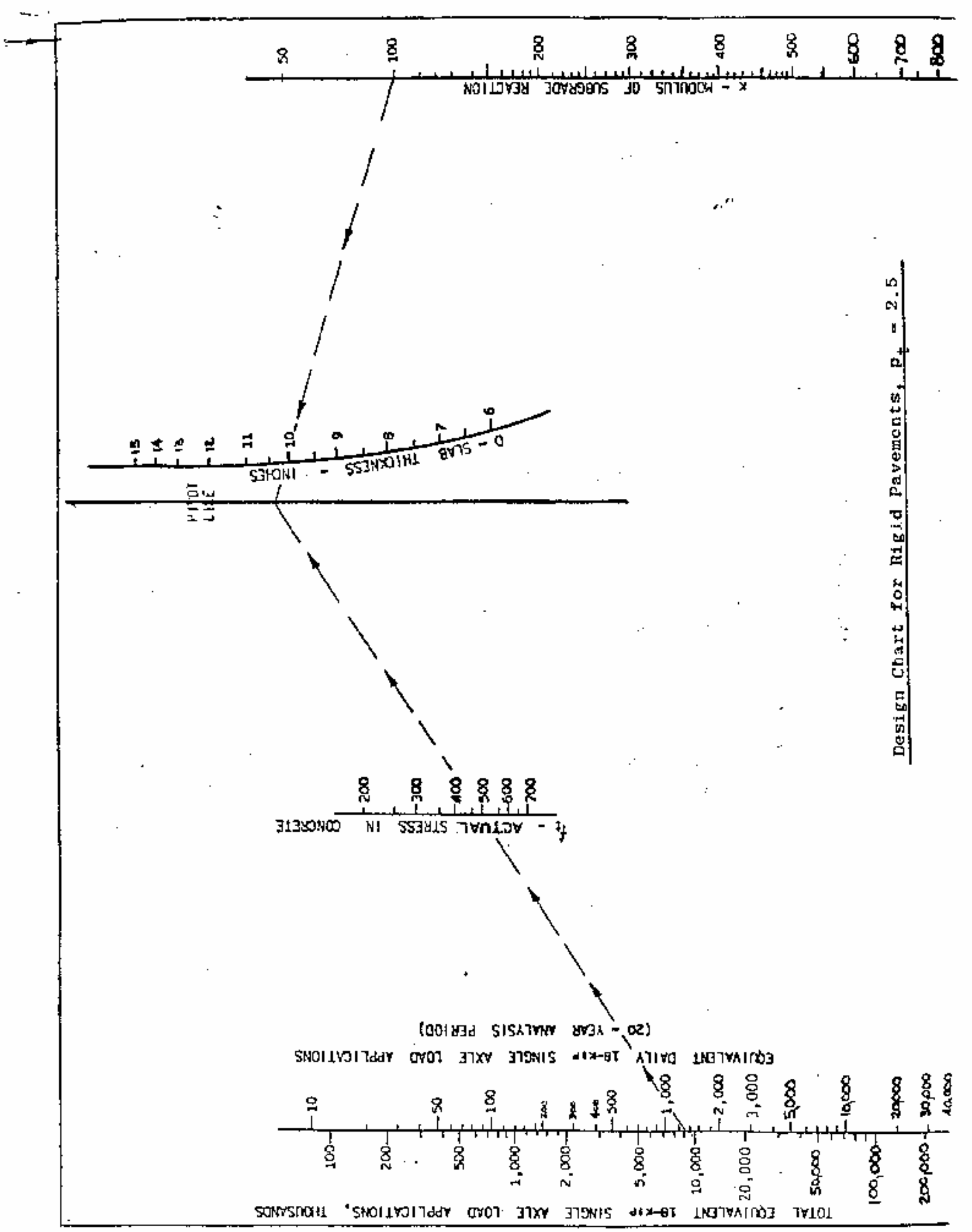


FIGURE 11.6 RIGID PAVEMENT FROM 1972 (ED.1981) ASHO DESIGN GUIDE

Valuable Design Features for PCC Pavements

The following considerations, when combined along with other design features contribute to added improvements in the performance of PCC pavements.

- Doweled Pavements

Based on the AASHO Road Test and subsequent tests the value of doweled transverse joints has been proven effective in load transfer. In addition, field test have shown that shorter joint spacing reduces the internal stresses in the concrete slabs. The Department had constructed most of the later original Interstates using 30 foot joint spacing and some sections contained dowels and others did not.

Some earlier un-doweled pavements have held up over 40 years without major faulting, others have not. Currently most PCCP is constructed with doweled joints at 15 to 20 foot spacing. If the proposed PCCP is adjacent to 30 foot slabs, then the joint spacing is reduced to 15 feet for the new construction. If the project is totally new construction, then the joint spacing is 20 feet as per Ga. Std. 5046H.

- Additional PCC Thickness

The previously described design proposed an 11 inch PCCP. If we constructed a 12 inch PCCP the average unit cost per square yard increases approximately $ 1.00 per SY. This additional inch of concrete pavement would carry approximately twice the loads and on heavily traveled roadways, the additional inch is something to seriously consider.

- Widened Outside Lanes

An outside lane, constructed 14 feet wide and striped at twelve feet, when used with a flexible shoulder, adds structure that reduces pavement deflections, thereby reducing stresses at the extreme PCC fibers. This feature improves pavement performance.

- Tied Concrete Shoulders

The use of tied concrete shoulders contributes to improved PCC Pavement performance in the same manner as a widened outside lane.

- Alternate Shoulder Designs for CRC Pavements

For future widening and economical design, the Department uses 13 foot outside lanes with the following shoulder alternates:

  - Roller Compacted Concrete (RCC), or
  - Asphalt Shoulders

- Designer Notes

As you work through the Rigid Pavement Design form, you record the modulus of subgrade reaction k, the modulus of subbase reaction k_1, trial depth of concrete pavement and the actual stress from the nomograph. As noted above, you should disregard a slab thickness that overstresses the concrete. Note your recommended pavement structure and include notes on joint spacing and the use of dowels.

Georgia Standard 5046 H provides the contractor with details regarding concrete pavement. Included in this is information regarding joint spacing and dowel bar sizes related to slab thickness.

Note: If you are proposing an undoweled concrete pavement, it shall be clearly noted on the design form and labeled on the typical section.

Limitations of the 1972 AASHTO Design Method

- It is essential that the user of the Rigid Pavement design procedure, whether by using equations or by using the nomographs, in the guide understand its limitations, which are:
- Although the traffic repetitions used in the development of the design relationship were experienced over only a two-year period, the traffic analysis period that must be selected for design is usually considerably longer than two years. The traffic analysis period should not be confused with pavement life, which is affected by other factors in addition to traffic.
- The design chart scales for working stress (f_t) in concrete and modulus of subgrade reaction (k) are derived from the Spangler modifications of the Westergaard theory of stress distribution in rigid slabs.
- There is no adjustment in the AASHO Road Test rigid pavement equation for an environmental or regional factor.
- Two major overall assumptions have been made in the development of these design procedures, as follows:
  - That the adequacy of the design will be established by soils and materials surveys and laboratory studies.
  - That the design strengths assumed for the subgrade and pavement structure will be achieved through proper construction methods.
- The pavement slabs at the AASHO Road test were 12 feet wide and did not benefit from shoulder support. Outside shoulders are typically constructed integrally with the slabs nowadays. This integral construction provides for added support to the slab edges. This additional support assists in load transfer and reduces slab stresses.
- Outside shoulders, are constructed full depth to use as a future lane. This full depth shoulder feature assists in load transfer and reduces slab stresses.
- Slabs in the outside lane are recommended to be constructed 14 feet wide, striped at 12 feet, when asphalt shoulders are proposed. This added width reduces slab deflections, and thereby reduces slab stresses.

References

1. Texas DOT Pavement Design Manual
2. 1972 AASHTO Interim Guide for Design of Pavement Structures
3. Concrete Pavement Fundamentals, ACPA, 2005

11.5.2 CRC Pavement Design Guidelines

A Continuously Reinforced Concrete Pavement (CRCP) is a Portland cement concrete (PCC) pavement that has continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints. A CRC pavement has the following features[1]:

- The pavement is allowed to crack in a random transverse cracking pattern and the cracks are held tightly together by the continuous steel reinforcement.
- Due to its high degree of continuity in its construction, a CRC pavement exhibits only minor cracking with good load transfer.
- This continuity, also provides for a more homogeneous load distribution onto the roadbed, and provides added capability in overcoming moderate foundation settlement.
- Edge conditions that occur in a Jointed Plain Concrete Pavement (JPCP) do not exist in CRC pavements.

Design Thickness[2]

From FHWA Technical Advisory T 5080.14, dated June 5, 1990, a CRCP slab thickness shall be designed to be generally as thick as JPCP slab thicknesses, unless local performance has shown that thinner pavements designed with an accepted design process to be satisfactory. In T 5080.14, there is also a background discussion for the rationale behind designing as full depth JPCP. GDOT designs CRCP thickness equal to JPCP.

Earlier CRCP Design Experience

Nationally, during the 1970's and early 1980's, CRCP design thickness was approximately 80 percent of the thickness of conventional jointed concrete pavement. A substantial number of the thinner pavements developed distresses sooner than anticipated. Attention to design and construction quality control of CRCP is also critical. A lack of attention to design and construction details has caused premature failures in some CRCPs.

The causes of early distress have usually been traced to: (1) construction practices which resulted in pavements that did not meet design requirements; (2) designs which resulted in excessive deflections under heavy loads; (3) bases of inferior quality, or; (4) combinations of these or other undesirable factors.

Longitudinal Reinforcement

In CRCP the function of the longitudinal reinforcement is not a structural one. Instead while the pavement is allowed to crack at random, steel keeps the cracks held tightly together.

A minimum of 0.6 percent (based on the pavement cross sectional area) is recommended to aid transverse crack development in the range of 8 feet, maximum, and 3.5 feet, minimum, between cracks. Deformed steel bars that meet the requirements set out in GDOT Specification 853, ASTM A 615 / ASTM 615M Grade 60 (420). The tensile requirements shall conform to the American Society for Testing and Materials (ASTM) ASTM A 615 / ASTM 615M Grade 60 (420).

GDOT targets 0.7 percent reinforcement in its design.

- Recommended spacing of the longitudinal steel is not less than 4 inches or $2\,{}^{1}/_{2}$ times the maximum sized aggregate, whichever is greater, and not greater than 9 inches.
- The recommended position of the longitudinal steel is between ${}^{1}/_{3}$ and ${}^{1}/_{2}$ of the thickness of the pavement as measured from the surface.
- The minimum concrete cover shall be $3\,{}^{1}/_{2}$ inches.
- The use of epoxy coated reinforcing steel is generally not necessary for CRCP.

Splicing - When splicing longitudinal steel, the recommended minimum lap is 25 bar **diameters** with the splice pattern being either staggered or skewed.

Transverse Reinforcement - GDOT uses #4 bars, grade 60 deformed bars meeting the same specifications as mentioned for the longitudinal reinforcement for transverse reinforcement. GDOT uses a 36 inch transverse bar spacing. The use of transverse reinforcing reduces the risk of random longitudinal cracks opening up and thus reduces the potential of punch-outs.2 The transverse bar spacing should be no closer than 36 inches and no further than 60 inches.2

Steel placement - Has a direct effect on the performance of CRCP. A number of States have found longitudinal steel placement deviations of ±3 inches in the vertical plane when tube feeders were used to position the steel. The use of chairs is required to hold the steel in its proper location. The chairs should be spaced and secured to prevent displacement such that the steel will not permanently deflect or displace to a depth of more than 1/2 the slab thickness.

Slab-Base Friction – The friction between the pavement and base plays a role in the development of crack spacing in CRCP. Most design methods for CRCP assume a moderate level of pavement/base friction.

Note: Polyethylene sheeting shall not be used as a bond breaker unless the low pavement/base friction is considered in design. Also, States have reported rideability and construction problems when PCC was constructed on polyethylene sheeting.

Base Design[2]

Free moisture in the base or subgrade has been identified as one of the major contributors that cause accelerated pavement distresses and failures. A well designed base shall provide a stable foundation and suitable construction platform critical for CRCP construction. Positive drainage considerations are recommended in the base design so that the base shall not trap free moisture beneath the pavement thereby undermining its performance. Bases that will resist erosion from high water pressures induced from pavement deflections under traffic loads, will act to prevent pumping. Stabilized bases should be considered for heavily traveled routes. Pavements constructed over stabilized or crushed stone bases have generally resulted in better performing pavements than those constructed on unstabilized gravel.

Joints

Longitudinal Construction Joints[2] - Longitudinal joints are necessary to relieve stresses caused by concrete shrinkage and temperature differentials in a controlled manner and should be included when pavement widths are greater than 14 feet.

Pavements greater than 14 feet wide are susceptible to longitudinal cracking. The joint should be constructed by sawing to a depth of one-third the pavement thickness. Adjacent slabs should be tied together by tie-bars or transverse steel to prevent lane separation. Tie-bar design is discussed in the FHWA Technical Advisory entitled "Concrete Pavement Joints."

Transverse Construction Joints[2] - A construction joint is formed by placing a slotted header-board across the pavement to allow the longitudinal steel to pass through the joint. The longitudinal steel passing through the construction joint is increased a minimum of one-third by placing 3-foot long shear bars of the same nominal size between every other pair of longitudinal bars.

No longitudinal steel splice should fall within 3 feet of the stopping side or closer than 8 feet from the starting side of a construction joint. If it becomes necessary to splice within the above limits, each splice should be reinforced with a 6-foot bar of equal size. Extra care is needed to ensure both concrete quality and consolidation at these joints.

If more than 5 days elapse between concrete pours, the adjacent pavement temperature shall be stabilized by placing insulation material on it for a distance of 200 feet from the free end at least 72 hours prior to placing new concrete. This procedure should reduce potentially high tensile stresses in the longitudinal steel. Special provisions for the protection of the header-board and adjacent rebar during construction may be necessary.

Terminal Joints / Anchors[2,3] - A terminal joint is used in continuously reinforced concrete pavement (see CRCP) to transition to another pavement type or to a bridge structure. They are found at the beginning and end of a CRC paving job, as well as spaced periodically in between, depending on the length of the job. Their function is to (1) isolate adjacent pavement types or structures, and (2) anchor the CRCP so that excessive movement does not occur. Terminal joints accommodate differential horizontal movements and prevent damage between a pavement and another pavement or structure. Because pavement performance can be significantly affected by the planned use and location of terminal joints, care should be taken in their design.[3] The most commonly used terminal treatments are the lug anchor which restricts movement, and the wide-flange (WF) steel beam which accommodates movement. GDOT has chosen the lug anchor as standard practice.[2]

Note: See Construction Details “CRC Pavement 12 inch Slab Thickness” and “5 Lug Terminal Anchor”. Currently not available electronically. Contact the Office of Road and Airport Design, Standards and Details Section at 404-656-5396, attn: Gary Owens.

Lug Anchor Terminal Treatment

Generally consists of three to five heavily reinforced rectangular shaped transverse concrete lugs placed in the sub-grade to a depth below frost penetration prior to the placement of the pavement. They are tied to the pavement with reinforcing steel[2].

Since lug anchors restrict approximately 50 percent of the end movement of the pavement an expansion joint is usually needed at a bridge approach. [2]

A slight undulation of the pavement surface is sometimes induced by the torsional forces at the lug, when 7 inch and 8 inch CRCP Pavements were used. This torsional end rotation has not been noted in thicker slabs and would cause no concern for CRCP slabs used by GDOT as those thicknesses are 12 inches. [2]

This lug anchor treatment relies on the passive resistance of the soil. It is not effective where cohesion-less soils are encountered. Figure 11.7 below shows a typical lug anchor terminal treatment.

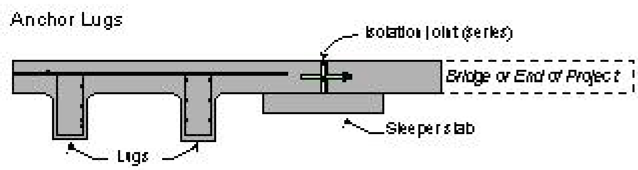


FIGURE 11.7 LUG ANCHOR

Leave-Outs [2]

Temporary gaps in CRCPs shall be avoided. The necessity for leave-outs is minimized by giving proper consideration to the paving schedule during project design. The following precautions can be specified to reduce distress in the leave-out portion of the slab in the event a leave-out does become necessary.

Leave-outs require 50 percent more longitudinal deformed bars of the same nominal size as the regular reinforcement. The additional reinforcement shall be spaced evenly between every other normal pavement reinforcing bar and shall be bonded at least 3 feet into the pavement ends adjacent to the leave-outs. All regular longitudinal reinforcement shall extend into the leave-out a minimum of 8 feet. Required slices shall be made the same as those in normal construction.

Leave-outs shall be paved during stable weather conditions when the daily temperature cycle is small. Because of the closeness of the steel extreme care shall be exercised in placing and consolidating the concrete to prevent honeycombing or voids under the reinforcement.

If it becomes necessary to pave a leave-out in hot weather, the temperature of the concrete in the free ends shall be stabilized by placing an adequate layer of insulating material on the surface of the pavement as described in paragraph 4e(3) (a) . The curing compound shall be applied to the new concrete in a timely manner. The insulation material shall remain on the adjacent pavement until the design modulus of rupture of the leave out concrete is attained.

Valuable Design Features

Ramps and Shoulders - PCC pavement for ramps and shoulders adjacent to CRCP is recommended because of the possible reduction in pavement edge deflections and the tighter longitudinal joints adjacent to the mainline pavement.

Ramps shall be constructed using jointed concrete pavement. The use of jointed pavement in the ramps will accommodate movement and reduce the potential for distress in the CRCP at the ramp terminal. When PCC pavement is used for ramps or shoulders, the joint shall be designed as any other longitudinal joint. Refer to Ga. Std 5046H for further information on joints.

Widened Lanes [2] - Widened right lane slabs shall be considered to reduce or eliminate pavement edge loadings when used with flexible shoulders. This is discussed in the FHWA Technical Advisory T 5040. 29,

A 2-foot integral widening of the mainline slab will reduce edge strains and deflections. To be effective, the travel lane shall be striped at 12 feet with the edge of the slab being moved into the shoulder and away from traffic load applications.

Paved Shoulders and Shoulder Type Construction[2] - It is recommended that the shoulder be constructed of the same materials as the mainline pavement in order to facilitate construction, improve pavement performance and reduce maintenance costs.

The use of full-width paved concrete shoulders is desirable. However, the additional cost of this design may not be warranted on all projects. In those cases, the use of widened lanes shall be given strong consideration. Widened lanes reduce edge stresses and the potential for edge drop-offs, increase safety, and reduce maintenance costs. A monolithic widening of 2 feet outside of the traveled way is recommended. Widened lanes are only effective when striped as 12-foot travel lanes. Consideration shall be given to the placement of rumble strips on the shoulder portion of the widened lane.

Typical GDOT CRCP Design

A typical CRCP design, currently used by GDOT, is summarized in the table below:

Pay Item Number	Material	Thickness (inches)	Spread Rate (lb/yd^{2})
-430-0820 Cl 1 430-1220 Cl HES	CRC Pavement	12	-
402-3190	19 mm SP At Mix Design Level "A"	3	330
310-1101	Graded Aggregate Base	12	-
430-0630	Reinforced. Concrete. Lug Anchors		

TABLE 11.6 CRC PAVEMENT DESIGN SUMMARY

Reinforcement - The longitudinal reinforcement shall consist of ASTM A615 Grade 60 size #6 reinforcing bars spaced at 5 inch intervals. The transverse reinforcement shall consist of ASTM A615 Grade 60 size #4 reinforcing bars spaced at 36 inch intervals.

Minimum and Maximum Rebar Concrete Cover - The concrete reinforcing cover is measured from the top of the slab. The reinforcing placement is summarized in the following table:

Material	Spacing, inches	ASTM A 615 Steel Grade	Bar Size	Min Concrete Cover	Max Concrete Cover
Longitudinal Reinforcement	5 inches C to C	60	#6	3 ½ inches	4 ¼ inches
Transverse Reinforcement	36 inches C to C	60	#4	4 ¼ inches	5 inches

TABLE 11.7 MINIMUM AND MAXIMUM REBAR CONCRETE COVER

Use of Widened Outside Slab - For long term pavement performance, it is also recommended to construct 14 foot wide outside lanes striped at 12 feet when used with flexible shoulders.

Reinforcement Ratio and Placement of Rebars - If a 12 foot wide slab is used, the clear distance of the first reinforcing bar from either slab edge shall be 4 $^{1}/_{8}$ inches. This provides a reinforcement ratio of 0.690%. If a widened slab is used, the clear distance of the first reinforcing bar from either slab edge shall be 3 $^{5}/_{8}$ inches. This provides a reinforcement ratio of 0.723%.

Shoulder Construction - It is additionally recommended that the shoulder be constructed full depth to match the mainline cross section for use as a future travel lane.

Concrete Shoulders

General

Concrete shoulders shall be tied to the mainline with properly spaced and sized tie-bars. Tied concrete shoulders will reduce pavement stresses and edge deflections. Tied concrete shoulders will also result in a tighter, easier to seal longitudinal joint that, when properly maintained, will effectively reduce water infiltration into the pavement structure.

Retrofitting tied concrete shoulders or lane widening will reduce edge stresses and deflections. The age, condition and remaining service life of the existing pavement play a significant role in determining whether a retrofit is practical. It is recommended that a retrofit be added only when an engineering and economic analysis indicates it to be a cost-effective solution.

Shoulder Thickness

Shoulders shall be structurally capable of withstanding wheel loadings from encroaching truck traffic. On urban freeways or expressways, the shoulders shall be constructed to the same structural section as the mainline pavement to ensure adequate load capacity at the interface between the mainline and shoulder; to provide for ease and economy of construction; and to prevent a "bathtub" condition under the pavement. This will also allow the shoulder to be used as a temporary detour lane during rehabilitation or reconstruction.

As an option for other than urban freeways and expressways, a tapered shoulder may be considered. Adjacent to the mainline, the shoulder shall be the same thickness as the mainline to permit mid-depth tie bar placement and to provide structural support for truck wheel encroachments. The shoulder may then be tapered to no less than 6 inches at the outside edge. Care must be exercised with a tapered section since a "bathtub" type condition can result, ponding/trapping water in the area of the lane/shoulder interface.

Subbase

It is recommended that the same type of sub base be used under the shoulder as under the mainline, especially on high-volume facilities. Care must be taken in designing the sub base cross-slope under concrete shoulders to avoid pocketing of water under the lane/shoulder joint and at the shoulder edge. Problems are often encountered at this location due to changes in sub base type, resulting in non-uniform support or difference in drainage characteristics.

References:

1. CRCP: A Long Lasting Pavement Solution for Today's Motorways, The Dutch Practice, by Marc J.A. Stet and Adrian J. van Leest, circa 2000. Technical Advisory T 5080.14, FHWA, Washington D.C., June 5, 1990
2. What are terminal joints, and why are they needed in continuously-reinforced concrete pavements (CRCP)?, by Steve Waalkes, *PE, Managing Director-Technical Services, ACPA*

11.6 Special Pavement Designs

11.6.1 Interstate Ramps

In the recent past, the Office of Maintenance had taken the lead in reconstructing Interstate Ramps statewide excluding those in urban areas. Their focus was to address the continuing maintenance of ramps and ramp shoulders. The Interstate ramps were asphalt or concrete but all the shoulders were asphalt. Typically the ramp shoulders were 6 to 8 feet wide and 3 3-1/2 inches thick over GAB or cement treated base. These ramp shoulders were, over time subjected to repeated truck loadings on a regular basis. These shoulders deteriorated over time. It was obvious that these shoulders were not designed for repetitive truck loadings. The Maintenance Office has reconstructed a substantial percent of the Interstate ramps statewide. The Maintenance Office decided that reconstruction of the ramps as PCCP would minimize their continuing maintenance efforts on this part of the Interstate system.

To simplify the ramp reconstruction process, Maintenance proposed one depth of section to be used statewide, 12 inch doweled PCCP over 5 inches of asphalt base over 12 inches of GAB for ramps and ramp shoulders. As Interstate interchange projects in the GDOT Construction Work Program moved into concept and design, the design offices followed the Office of Maintenance Ramp as typical.

The Pavement Design Committee reviews proposed designs for projects prepared in the design offices, Road, Urban and Consultant Designs. OMR works closely with Maintenance on their major resurfacing/reconstruction projects. The design used by Maintenance took the best case/worst case approach. How thick was the thickest PCCP in Georgia and what base and subbase gives us the best support for the PCCP. Traffic, loadings, soil conditions were not considered in depth. As designed projects moved through the Pavement Design Committee, the question was asked "Based on the traffic and soil conditions how thick should the ramp paving be versus what you have proposed?" The Pavement Design Committee agreed that a ramp design should propose a PCCP thickness based on the traffic and soil conditions and that a ramp pavement design for the 12-5-12 would also be submitted. The consensus of the Committee was, there would be discussion at the meeting considering both designs and recommendation made.

Since the 12-5-12 section was proposed, OMR has evaluated the 5 inch asphalt base (25mm Superpave) and recommended that 3 inches of 19 mm Superpave be used.

Note: Figure 11.8 shows 2 feet inside shoulder (left side of direction of travel) and 10 feet outside shoulder (right side of direction of travel). The AASHTO Guidelines, "A Policy on Geometric Design of Highways and Streets" (Green Book), 12 feet is recommended as the total shoulder width (left + right). GDOT has appealed to the FHWA for approval of variations of the shoulder width. Staging traffic, future use of shoulder, type of ramp, and possible lane additions may influence a variation of the 2 feet shoulder-16 feet lane-10 feet shoulder ramp typical section.

11.6.2 High Stress Areas

Superpave Mixes are more rut resistant than conventional asphalt mixes, which we no longer construct. The addition of Gilsonite modifier to Superpave increases the stiffness, reduces rutting and increases cost per ton of Superpave. The cost for the addition of an asphalt modifier and the small volume of modified mix equates to a higher cost /ton.

High stress areas occur throughout the state and are most common at intersections and at ramp terminals. The effort by Maintenance has corrected most ramp rutting problems by reconstructing the ramps as PCCP. We have let projects with modified asphalt to reduce rutting as well as tried UTW (ultra thin white-topping) and even full depth PCCP. These are all viable options that come with varying costs. OMR has taken the lead with input from Maintenance and Construction on this issue. What we are focusing on is to provide a smooth riding pavement and reduce the inconvenience to the motoring public to drive on a smooth riding intersection.

At this point in time, use the following approach. If the design proposes to construct a PCCP Interstate ramp, the cross road paving should in PCCP shall be considered as well. Same approach applies to preparing a PCCP slab thickness based on traffic and soil conditions for the ramp and cross road as well as proposing the 12-3-12 section. The Pavement Design Committee will review, discuss and approve the PCCP depth of section. On roadway projects that propose a flexible section and contain intersections that exhibit rutting and shoving, rely on the pavement evaluation or your on-site inspection to request/recommend a UTW at high volume/stress intersections. OMR will provide guidance and recommendations for possible UTW. Typically the limits of full depth PCCP or UTW would be the radius returns. Specific site conditions will dictate the limits of PCCP or UTW.

11.7 Minor Project Materials and Designs

Intersection Improvements - Intersection improvement projects do not require the approval of the Pavement Design Committee. Submit designs to OMR for review and approval. Depending on the project and site specific conditions, UTW or PCCP may need to be considered, if this is the case contact OMR and request their input.

Note: See the letter from Buddy Gratton, P.E., Director of Preconstruction, dated June 7, 2005, "Standard Pavement Sections for Minor Projects", on the Office of Consultant Design and Program Delivery's web site at: http://www.dot.state.ga.us/dot/preconstruction/consultantdesign/design/pave-minor.pdf.

Bridge Replacements - Bridge replacement projects vary in size and scope in regards to the amount of pavement associated with them. Some of these projects stand alone others are associated with programmed roadway widening/construction projects. Those associated with roadway projects shall have their pavement design submitted to the Pavement Design Committee for review and approval. The stand alone bridge replacements need to have pavement cored and evaluation prepared by OMR. The recommendations from OMR should be considered in the design of the pavement. The proposed designs should comply with Buddy Gratton’s letter as noted above. The pavement design shall be submitted to OMR for review and approval.

Passing Lanes - Passing lane projects are District projects for the most part. The existing pavement shall be cored and have a pavement evaluation prepared by OMR. The proposed designs should comply with Buddy Gratton’s letter as noted above. The pavement design shall be submitted to OMR for review and approval.

Turning Lanes - What is a turning lane project? We all know what a turn lane is but is it a right turn lane or a left turn lane? Typically these turn lanes are part of a programmed project, whether a roadway widening or intersection improvement.

There have been programmed projects that have upgraded older 4 lane divided highways to current median guidelines, typically reconstructing median openings from type “A” to type “B”. Pavement cores and pavement evaluations shall be requested through OMR. The proposed designs should comply with Buddy Gratton’s letter as noted above. The pavement design shall be reviewed and approved by OMR.

The other turn lane is the one associated with a driveway permit. Driveway permits are processed through the District Access Engineer. The permit contains a proposed typical section for what is proposed to be constructed within State R/W. If the proposed development is within a programmed project, the driveway permit shall be submitted to the design project manager by the District Access Engineer for review and comment. The design project manager shall review the proposed paving section and make recommendations accordingly.

11.7.1 Temporary Pavement Materials and Design

This section applies to temporary flexible pavement designs and materials required for construction staging or detours meeting one of the two following cases:

- The temporary pavement will be removed during construction when no longer needed.

- A portion or the entire temporary pavement will become part of the permanent (20 year design period) flexible pavement section being constructed. Typically in these cases additional asphalt lifts are overlaid on the temporary flexible pavement to construct the permanent pavement section.

Temporary Pavement Design

Ga. Std. 9109 provides a typical section with a temporary pavement section for a construction detour to cross over a depressed median. The designer is cautioned that the Standard's temporary pavement section may not be structurally sufficient when high volume traffic or high truck percentage of total traffic exists or the detour may be in operation for a long period of time. The designer will include a copy of the detail in the project's pavement design package submitted to the Pavement Design Committee for approval if they propose using this temporary flexible pavement section.

The designer will prepare temporary flexible pavement designs for those applications where GDOT Standard 9109 is not appropriate. Temporary flexible pavement design follows the same processes, guidelines, and procedures in this manual for permanent flexible pavement sections. The designer will include all temporary pavement designs they propose in the project's pavement design package submitted to the Pavement Design Committee for approval. Some specific guidelines for temporary flexible pavement design are given below.

Traffic Data

The temporary flexible pavement design period is the number of years the pavement will be used for construction staging or detour. The project's Traffic Data provides the initial year one-way AADT. The designer will calculate the final year one-way AADT using a yearly simple growth rate calculated from the project's initial and final year AADTs. Apply this rate to the initial year ADDT to calculate a final year one-way AADT corresponding to the temporary pavement's design period.

Lane Distribution Factor (LDF)

The designer will use a LDF for the percentage of trucks in the design lane that is appropriate for the specific temporary pavement use.

Terminal Serviceability Index

- Use 2.0 when the temporary pavement will be removed during construction when no longer needed.
- Use 2.5 when a portion or the entire temporary pavement will become part of the permanent flexible pavement.

Required and Proposed Structural Number (SN)

Under design by 10% to 15% temporary flexible pavement that will be removed.

Under design by 0% to 5% temporary flexible pavement that will become part of the permanent flexible pavement. In some cases it may not be possible to under design a temporary flexible pavement section by this amount or even over design may be required. One possible case is when the temporary flexible pavement section may need to be thicker than that required for a 0% to 5% under design to match into existing or new construction pavement sections during staging.

Temporary Pavement Materials

Temporary flexible pavement design follows the same Superpave mix type and design level guidelines used for permanent flexible pavement with the following additional guidance provided:

- Temporary flexible pavement surface layer is normally 9.5 mm or 12.5 mm Superpave. 19 mm Superpave may be used as the surface course when necessary.
- OGFC and PEM Superpave mix types are not required in the temporary flexible pavement surface.
- Superpave mix design level A may be used in all layers unless otherwise specified in the project's construction plans.

11.8 Overlay Pavement Design

An existing pavement structure becomes an overlay candidate when the following conditions exist:

- The load carrying capacity of the existing pavement needs to be increased.
- Pavement rehabilitation is required to extend the useful life of an existing pavement.
- An adjacent lane such as a passing lane or a turn lane is being added, and the existing lane may suffice with a new surface overlay.
- To correct surface deficiencies of the existing pavement.

11.8.1 Flexible Pavement Overlays

Flexible Pavements Overlays can be broken down in two major categories.

Functional Overlays

These are typically minor rehabilitations that correct surface deficiencies, extend the useful life of a flexible pavement, and restore the pavement riding surface to an acceptable standard.

Those are determined by The Office of Maintenance, and generally consist of a thin hot mix overlay for a flexible pavement.

If an existing lane is overlaid so that its profile is raised up to the same grade as a new turn lane addition or a passing lane addition, and if additional structure is not needed is considered a functional overlay as well.

Structural Overlays

These are used to increase the load carrying capacity of the existing pavement or extend the useful life of an existing pavement. Those are accomplished by as simply as a surface layer addition, or may be more involved, in that milling the existing pavement to a certain depth is needed prior to adding any new layers to restore the projected structural needs of the pavement. Use the following guidelines when considering structural overlays:

- On widening projects, if any portion of the existing pavement is to be overlaid, then consideration should be given as to the length of each individual segment being retained and their function in the widened pavement structure.
- For constructability considerations, a general rule of thumb is that if the segment is shorter than 1000 feet, then reconstruction is more suitable.
- If the retained portion of the existing pavement will carry mainline traffic, then it shall be designed using the same standards (underdesign percentages) as new construction.
- The thickness of the overlay is determined from the WIN_APD software program, which solves the 1972 Flexible Pavement Design Nomograph using a personal computer.
- In addition to those options, Ultra-Thin Whitetopping may be an option where the area being addressed has severe rutting, and sufficient asphalt structure to carry the anticipated loading. Contact OMR regarding UTW applications.

11.8.2 Rigid Pavement Overlays

GDOT has overlaid many of its original PCC pavements with a Flexible Pavement Structure. The resulting pavement is termed a composite pavement because of the different materials that make it up.

The use of composite pavements has provided GDOT with mixed results. Because of this, GDOT does not currently use flexible overlays over rigid pavements.

Maintenance Treatments

Rehabilitations that add strength to and extend the service life of rigid pavements, include but are not limited to dowel bar retrofit and slab replacements. Those are also determined by the Office of Maintenance.

Structural Overlays

The Department currently uses several overlay treatment alternates that add structure to existing rigid pavements. Reference 1 below lists several overlay treatments that have been used. All overlays considered by GDOT are considered to be unbonded overlays.

Ultra-Thin Whitetopping

Ultra-Thin Whitetopping is a PCC pavement, 3 to 4 inches in thickness that is used to address rutting problems in high stress flexible pavement intersections.

A crucial element in the design of a UTW overlay is leaving sufficient flexible pavement thickness after milling and correcting the surface deficiency. Its design is based on fatigue failure and is detailed in Chapter 6 of Publication No. FHWA-IF-02-045.

Whitetopping

If the PCC pavement being used, has a calculated thickness from 4 to 8 inches, and whether that pavement is used to replace either an existing pavement that is in poor shape such as in a turn lane, or is used as a new pavement then its is called a Whitetopping pavement. This pavement type is designed as a standard PCC pavement.

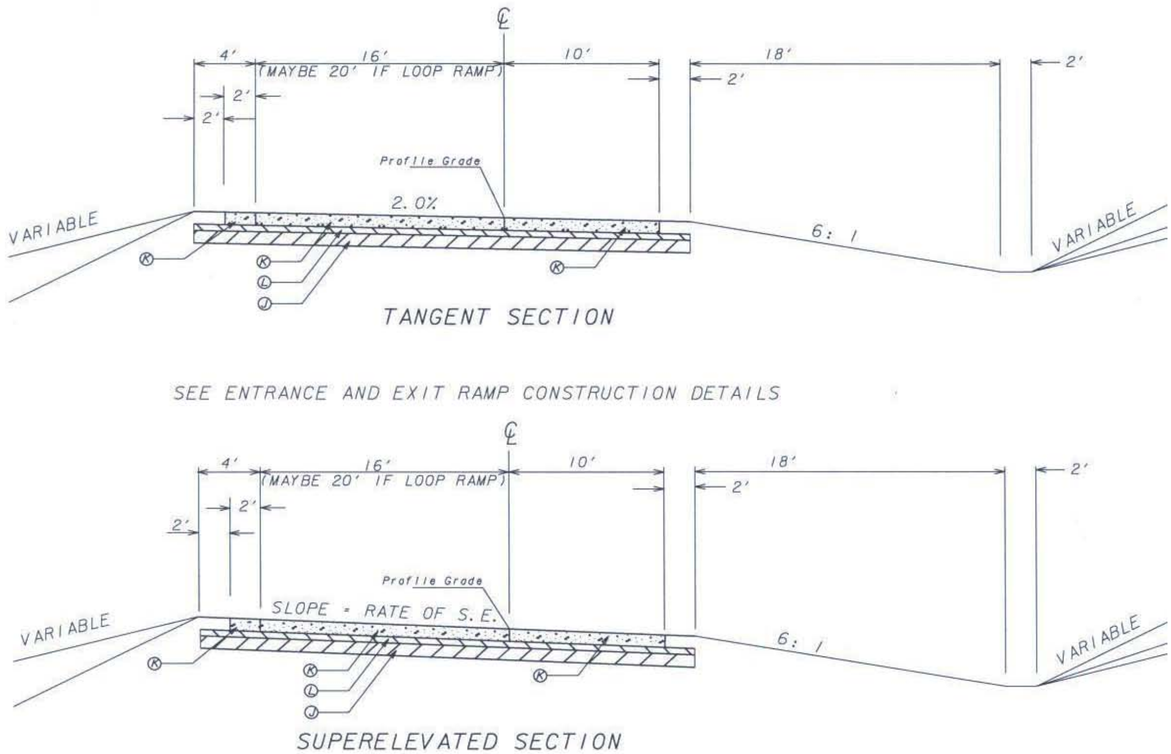
Unbonded PCC Overlays

An unbonded PCC overlay is a PCC overlay of an existing composite pavement. This type of overlay allows the use of the existing pavement as a base for building the overlay. This pavement type is designed as a standard PCC pavement.

Alternately, an existing flexible pavement that is a suitable candidate for full depth reconstruction may be an unbonded PCC overlay candidate. This recommendation may be made if enough existing pavement thickness exists that will allows the milling of deteriorated layers and placing the new PCC pavement on top.

References

1. *Portland Cement Concrete Overlays*, Publication No. FHWA-IF-02-045, Federal Highway Administration, Washington, D.C., 2002.



TS-4

VERIFY THESE TYPICALS AND NOTES ARE STILL APPLICABLE
BY REFERRING TO THE DEPARTMENT'S DESIGN MANUAL

** VARIABLE SLOPE CONTROLS		
SLOPE	FILL	CUT
6:1	3'	--
4:1	3'-10'	--
2:1	OVER 10'	ALL

Ⓚ PLAIN PORTLAND CEMENT CONCRETE PAVEMENT, 12 In.

Ⓛ 9 mm SUPERPAVE ASPHALT CONC. BASE (330 lb/yd^2) SUPERPAVE (MIX DESIGN LEVEL according to 9/05/01 guidance)

Ⓙ GRADED AGGREGATE BASE, 12 In.

* 2:1 SLOPES REQUIRE GUARDRAIL

FIGURE 11.8

12 Preservation, Rehabilitation, Restoration

12.1 Background

Pavement maintenance can be grouped into two categories: preventive and corrective maintenance. Preventive maintenance applications prevent the development of distresses or they reduce the rate of distress development in the pavement structure. In general, preventive maintenance applications are designed to preserve the structural capacity of the pavement. They do not intend to increase the structural capacity of the pavement. Preventive maintenance applies lower-cost treatments to retard a highway's deterioration, maintain or improve the functional condition, and extend the pavement's service life. With various short-term treatments, preventive maintenance can extend the pavement's life by an average of 5 to 10 years. Applied to the right road at the right time – when the pavements are mostly in good condition – preventive maintenance can significantly improve the network condition at a lower unit cost. Corrective maintenance applications are designed to restore distressed areas to an acceptable level. Corrective maintenance applications increase the structural capacity of the pavement.

12.2 Georgia Pavement Preservation Program

The State Maintenance Office believes that having the best maintained highways begins during the preconstruction stage. Proper soil analysis must be performed for a pavement design to be determined. A designer must implement the proper design features that are functional and maintainable. Construction staff must carry out project inspection and quality control. Then maintenance can carry out its pavement preservation program. All of these steps provide a safe, efficient, and sustainable highway system for the traveling public.

Outlined below is a brief summary of the pavement preservation program GDOT utilizes, in no particular order:

- Pavement condition evaluation performed annually of all state highways both asphalt and concrete pavement.
- Goal of clipping half of all state highway shoulders annually.
- Annual comprehensive inspections performed denoting deficiencies of the highway, inclusive of entire rights of way features (vegetation, signs, guardrail, pavement markings, etc.).
- Biennial inspection of minor drainage structures with span openings less than 20 feet.
    - Crack filling.
    - Strip sealing.
    - Spot overlays.
    - Pothole and deep base repair.

- Resurfacing program that has a goal to resurface 10% of all state highways annually based on need. The resurfacing treatment is typically less than 2 inches (in thickness) of hot mix asphalt.
- Chip seal the routes that meet the Department's criteria (ADT less than 1500, minimal pavement distresses, etc.).

12.3 Process for Resurfacing Georgia State Highways

Every mile of Georgia state highways are evaluated annually using the pavement evaluation condition system (PACES). This evaluation is performed by local DOT Area Maintenance Managers. For routes that receive a rating of 75 or below (scale rating of 0 to 100 with 100 being a newly resurfaced highway), they are then reviewed by the Assistant District Maintenance Engineer, and then by a representative form the State Maintenance Office. Once the District Office and the State Maintenance Office concur that the highway warrants to be submitted to be resurfaced, the process of preparing a resurfacing project begins. Outlined below is a simplified summary of how a state highway is submitted and prepared to be let to construction for resurfacing work.

PACES is designed to indicate the amount and type of surface distress on a roadway at the time the survey is made. The system standardizes the terminology for the types of defects that can be found on a pavement in Georgia and defines the various levels of severity for these defects. This system will allow roads to be rated objectively.

This system only addresses the structural condition of the pavement surface and does not include skid resistance and ride-ability both of which are measured with high speed testing equipment.

A number of distresses, which relate to the pavement performance, have been identified for flexible pavement and surface treatment. Both the presence of these distresses and the severity levels must be taken into account when rating a pavement. These distresses are as follows (see distress definitions):

- Rut Depth
- Raveling
- Load Cracking
- Edge Distress
- Block Cracking
- Bleeding/Flushing
- Reflection Cracking
- Corrugations/Pushing
- Patches and Potholes
- Loss of Section

There are other types of defects which are not being considered because they occur infrequently or because they are included in one of the above categories at a certain severity level. Transverse cracking, for instance, is considered to be an initial stage of block cracking and is therefore rated in that category.

Ratings are done for each mile (or partial mile) by selecting a sample section that is representative of the pavement condition for that rating segment. The defects noted for each rating segment within a project are then averaged to obtain the representative pavement condition for that project. A project rating is then determined from deduct values, which have been established for each defect and severity level.

12.4 Distresses Measured

12.4.1 Rutting

Rutting is a permanent, longitudinal depression that is greater than 20 feet in length that forms under traffic loadings in the wheelpaths. It can be caused by insufficient compaction, plastic movement of the mix, or an unstable foundation. Rut depths are estimated in both wheelpaths in the sample area and recorded on the survey in units of 1/8 of an inch. If rutting is extensive (more than 3/8 inch), then actual measurements may be necessary.

According to its definition, the following questions may be addressed when identifying rutting distress:

1. Compared with the pavement outside the wheelpaths, is there any depression deformation in the wheelpaths?
2. Is this deformation in the longitudinal direction?
3. Is there any cracking in the longitudinal direction which is associated with the deformation in the wheelpaths?

12.4.2 Load Cracking

This type of cracking is caused by repeated heavy loads and always occurs in the wheelpaths. It usually starts as single longitudinal crack in the wheelpaths. As progression continues, short transverse cracks occur that intersect the original longitudinal cracks. Additional longitudinal cracks occur in the wheelpaths. As the number of longitudinal and transverse cracks in the wheelpaths increases, polygons are formed by the intersection of these cracks. As deterioration continues, these polygons become smaller (due to additional cracking) and, in the worse case, begin to pop out. When load cracking progresses to the point where small polygons are formed, rutting can become extensive and pumping of base material can occur.

12.4.3 Block/Transverse Cracking

This type of cracking is caused by weathering of the pavement or shrinkage of cement-treated base materials. Block/transverse cracking is not load related. The block pattern is distributed uniformly throughout the roadway and not concentrated in the wheelpaths. Block cracking is interconnecting cracks forming a series of large blocks usually with sharp corners.

Block/transverse cracking begins as single, tight transverse, longitudinal or combinations of both types of cracks. In the beginning, block/transverse cracks may not form a recognizable block pattern, just longitudinal and/or transverse cracks that are not associated with the wheelpaths.

As this type of cracking progresses, a definite block pattern occurs, and the cracks become wider. As the cracking worsens, the block pattern densifies (small blocks), and/or the cracks become very wide (> 1/8 inch).

12.4.4 Reflection Cracking

This type cracking is caused by the "reflection" of joints and cracks through an asphaltic concrete overlay from the underlying PCC concrete pavement, and occasionally from cement-treated bases, especially ones with high cement contents and/or thin asphaltic concrete overlays. These reflection cracks begin as tight cracks and progress to very wide cracks with spalling. Transverse cracks will be at right angles across the width of the roadway in a repeated pattern (i.e., every 30 feet). Longitudinal cracks will normally be fairly straight, continuous cracks near the pavement edge associated with the underlying edge of a narrower PCC concrete pavement, which has widened and overlaid with asphaltic concrete. Any other "related" cracks will be associated with failures in the underlying PCC concrete pavement and will reflect the size and shape of such failures.

12.4.5 Raveling

This condition is the progressive disintegration of the pavement surface. It is caused by traffic action on a weak surface. Aggregate particles become dislodged from the binder and this loss of material can progress through the entire layer. Raveling ranges in severity from the loss of a substantial number of surface stones to the loss of a substantial portion of the asphalt surface layer. For purposes of rating, a slurry seal that has "peeled off" is considered Level 3 Raveling.

12.4.6 Edge Distress

Edge distress is cracking and pavement edge break-off within 1 to 2 feet of the pavement edge and not associated with the wheelpath area. The cracking can be in the form of longitudinal, transverse, or in many instances alligator-type cracking. It may sometimes be difficult to distinguish between alligator cracking in the wheelpath and along the edge of the pavement, especially on narrow pavements. It must be called load cracking when it occurs in the wheelpath. It can not be called both load cracking and edge distress.

Timely preventive maintenance and preservation activities are necessary to ensure proper performance of the transportation infrastructure. Experience has shown that when properly applied, preventive maintenance is a cost-effective way of extending the service life of highway facilities and therefore is eligible for Federal-aid funding. By using lower-cost system preservation methods, states can improve system conditions, minimize road constructions impacts on the traveling public, and better manage their resources needed for long-term improvements, such as reconstruction or expansion. Preventive maintenance offers state DOTs a way of increasing the return on their infrastructure investment.

GDOT's maintenance program is nationally recognized for maintaining quality pavements on the Georgia State Highway System. The maintenance program emphasizes treating pavements before they have deteriorated to the point of needing reconstruction. To help achieve this result, the maintenance program relies on quick delivery of maintenance projects. This quick delivery is possible because maintenance projects follow an accelerated preconstruction process that is exempted from the requirements of GDOT's Plan Development Process (PDP).

12.5 Asphalt Pavements

Preventive Maintenance (PM): PM on asphalt pavements is typically limited to such categories as crack sealing, joint sealing, slurry seal, chip seal, etc. Asphalt resurfacing to correct *surface* problems (raveling, rutting in the surface layer, water intrusion, low friction, minor surface cracking, etc.) is PM because its sole purpose is to restore the functional aspects of the pavement. Resurfacing up to 2.0 inches on secondary roads and up to 3.0 inches on interstate highways is considered PM.

Reconstruction: Any asphalt treatment that requires the removal of the entire pavement structure all the way down to the base course would be considered reconstruction.

Restoration: Any asphalt pavement treatment that does not fall under either the PM or Reconstruction categories would be considered restoration. In most cases, the definition of restoration for asphalt pavement would be triggered when a mill and inlay had to go deeper than the top layer of dense-graded mix, but not as deep as the base course.

12.6 Treatment Selection

Because of the State's commitment to preventive maintenance, GDOT is able to apply non-structural repairs to the pavement sections that fall below a 70 each year. In order to determine the appropriate treatment strategy, overall ratings are reviewed and then individual distresses are evaluated to determine the recommended treatment. The State Maintenance Office does treatment selection with input from the District Maintenance and Area Maintenance Engineers in cases where unusual problems exist, such as cross-slope problems that must be addressed. Some additional testing may be conducted if there is concern about the structural adequacy of a pavement, but most of Georgia's roads do not have the severity of conditions that would signal the need for this level of repair. The guidelines in Table 1 are used for treatment selection.

Treatment	Condition Rating/Distress Information	In-house or Contract Forces
Crack Sealing/Joint Sealing	75-80	In-house maintenance forces
Seal Coats	70-77	In-house maintenance forces
Spot Overlays	70-80	In-house maintenance forces
Deep Patching	Localized Subgrade problem	In-house maintenance forces
Milling, Thin Overlay, Mill and Inlay	<70	Contract forces
Concrete Pavement Restoration	<70	Initially in-house forces, now more contract forces

TABLE 1. GDOT GUIDELINES FOR MAINTENANCE TREATMENT SELECTION

Average daily traffic (ADT) levels are a factor in treatment selection. For pavement sections with higher ADTs, the Department is more inclined to recommend overlaying the roadway. If fatigue cracking is present in a section eligible for an overlay, a mat may be placed prior to the overlay to retard reflection cracking. Pavement sections with an ADT greater than 1500 or within the city limits are not eligible for a chip seal treatment.

13 Pavement Design and Approval Process

13.1 Classify Project and Approval Process

There are two options available in the Pavement Design and Approval Process. First you must classify the project as Major or Minor, using engineering judgment if special circumstances exist.

13.1.1 Major Projects

Use the following guidelines to determine whether or not the project is a major project:

- Projects generated in Consultant Design, Road Design or Urban Design.
- On-System projects regardless of ADT except as noted below.

Submitting Designs For Approval

Do the following to submit your major project design for approval:

1. Select a design (consult with office representative to the PDC for possible alternatives).
2. Complete the checklist.
3. Submit designs and typical sections to the representative of the PDC for review and submission to the PDC.

13.1.2 Minor Projects

Use the following guidelines to determine whether or not the project is a minor project:

- Off-System roads less than 4000 ADT
- Bridge replacements, intersection improvements, passing lanes and turning lane projects. See letter on Office of Consultant Design and Program Delivery web site at

  http://www.dot.state.ga.us/dot/preconstruction/consultantdesign/design/pave-minor.pdf

Submitting Designs For Approval

Do the following prior to submitting your minor project for approval:

- Select a design (consult with office representative to the PDC for possible alternatives).
- Minor Projects that exceed the above traffic constraints will require review and approval from the State Pavement Engineer.

13.1.3 Submittal

The Pavement Design Committee Meetings' schedule will be made available by each offices' PDC Representative, and it is the Project Manager's responsibility to submit designs as soon as is necessary. Ideally, designs should be submitted 6-9 months prior to Letting, and to the PDC Representative 3 weeks prior to a scheduled meeting. You will need an approved design to go to FFPR, and must have both an approved soil survey report and existing pavement analysis (if you have an overlay section) before submitting designs. The total process will require one to one and a half year advance planning.

Note: Major or Minor designation is <u>NOT</u> the same as PDP designation of Major or Minor. Major projects assigned to the District Offices should be submitted to OMR attention Pavement Management Branch for review and approval to the State Pavement Design Engineer.

13.1.4 Pavement Design Committee and Approval Process

- PDC Members, Bylaws, and pavement design submittal guidance can be found on the Department's web site at http://www.dot.state.ga.us/topps/index.shtml .
- TOPPS 5560-1 and 5560-2 should be routinely checked for revisions.

Excerpt From TOPPS 5560-1, Pavement Design Committee Members

The following Department and FWHA personnel are to constitute a Committee on Roadway Pavement Structures:

- State Pavement Engineer (Chairman)
- State Pavement Design Engineer (Secretary)
- Construction Office Representative
- Maintenance Office Representative
- Road and Airport Design Representative
- Urban Design Representative
- Engineering Services Representative
- FHWA Representative
- Office of Consultant Design and Program Delivery

Excerpt From TOPPS 5560-2, Bylaws

1. Each member shall designate an alternate who may serve in his place at the member's option. In case of a meeting where the member is not present, the Chairman can call upon the alternate to serve in the absence of the member. The names of those alternates shall be filed with the Chairman and the Secretary. Alternates are encouraged to attend all meetings.
2. Meetings shall be held quarterly when called by the Chairman at the time and place designated by the Chairman. The Chairman may call a Committee Meeting upon the request of any member. The requesting member shall inform the Chairman, in writing, of the matter he wishes to bring before the Committee.
3. Copies of all project pavement design analyses to be considered at the Committee Meeting shall be furnished to each member and his alternate at least one week before the date of the Committee Meeting.
4. Copies of all other proposals which are to be presented to the Committee, shall be furnished to the member or his alternate at least one week before the date of the Committee Meeting.
5. A quorum shall consist of five members or alternates. If possible, the member presenting a proposal to the Committee shall be present.
6. The minutes of the meeting shall be recorded and signed by the Secretary. A file of the minutes shall be maintained by the Secretary.
7. A typical section or sketch of each new design shall be available at the meeting at which the design is considered. These typical sections or sketches shall be furnished by the member responsible for the design.
8. Recommended changes in roadway pavement structure designs on projects under contract shall be submitted to the Committee by the State Construction Engineer in the same manner that new projects are submitted by the State Consultant Design Engineer, State Urban Design Engineer, and the State Road and Airport Design Engineer.

District Pavement Design Approval Process

1. District pavement design and pavement type selection processes should be similar to Figure 13-1.
2. District Project Manager prepares pavement design package and submits to District Quality Control and Review.

3. District Engineer submits the Pavement Design Package to the State Pavement Engineer. Normally these reviews will be approved outside of the normal Pavement Design Committee Meetings. The District Pavement Designs may be presented to the Pavement Design Committee by the State Pavement Engineer for purposes of discussion or entry into minutes.
4. Pavement Design approval or recommended changes are returned to the District Engineer for filing. Approved designs are submitted with subsequent field plan review requests to Engineering Services by the District Engineer.

13.2 Preparing a Typical Section

This is general guidance only. These guidelines should be used in conjunction with the Design Policy Manual, with preference being given to the Design Policy Manual in the case of conflicting guidance.

1. Typical sections should reflect the "typical" situation. Being too specific can clutter the typical. Adding a note to "see plan sheet" will accomplish the same result.
2. Label and number (TS-1, TS-2, and so on) each typical. (Tangent and S.E.)
3. Scale the typical, but note it as N.T.S. (Not To Scale)
4. Show all dimensions.
5. Stationing should not be separated for the tangent section and the super-elevated section. Stationing should be shown only on the tangent section with a note under the superelevated section referencing to the stationing of the associated tangent section. If a transition occurs through a superelevated section then a separate section may be warranted and stationed accordingly. S.E. should begin and end at the transition station. Show slope controls.
6. Place one guardrail detail on the first applicable typical section. Utilize the miscellaneous typical section detail sheets in lieu of repeating various cell details on the typical section.
7. Show the location of the profile grade and S.E. rotation point.
8. When super-elevation is present, show a maximum roll-over (break-over) table between the mainline and the shoulder. The maximum roll-over is 8%. The preference is to omit a table and refer to Georgia Standard 9028C, but until the Standard is corrected and updated shoulder break-over can be shown in table or with general note.
9. Label or flag all materials. (Asphalt, GAB, ground-in-place rumble strips, curb and gutter, and so on).
10. For asphalt pavements show materials as spread rates, lbs/sq yd. GAB is shown as a thickness in inches. If paving material is supplied from

Florida or soil cement, top soil or other soil bases are used, then show a Square Yard measure for payment if weighing is inconvenient. If Square Yard measurement is specified, the typical section shall be clearly dimensioned along the top finished surface and dimensions noted as the "Width of Payment."

11. Indicate cross-slope, with direction arrows pointing down the slope.
12. Apply rumble strips where appropriate.
13. Normal cross-slope on roadways is 2%.
14. Normal outside rural shoulders will be sloped at 6% for the full width including both paved and graded shoulders. Inside paved shoulders, 2 feet or less shall match the normal paving slope, full width paved or graded inside shoulders shall be sloped 4% into the median.
15. If the typical is drawn to a large scale and some areas are illegible, show a detail.
16. Materials shall match the materials proposed on the Pavement Design.
17. Indicate the Construction Centerline location.
18. Design a shoulder depth of paving compatible with the travel lanes. Currently the Department's policy is that the outside shoulder will be a reduced depth compared to the travel lanes. As a result of special conditions, discussions with District Offices or Construction Office it may prudent to make the outside shoulder full depth matching the travel lanes, but pay attention to the differences of the asphalt design level (see http://www.dot.state.ga.us/dot/preconstruction/consultantdesign/design/superpave.pdf) as it may vary between the surface courses.
19. An urban shoulder shall be in accordance with the sidewalk details (see Georgia Construction Details A-3 and the Design Policy Manual).
20. The allowable range table will be shown on typical sections where an overlay is required.
21. See Georgia Construction Detail S-8 for additional consideration of bike lanes.
22. Include all applicable details such as Class B paving detail, pavement fabric detail, guardrail detail (urban/rural shoulder or interstate/PCC shoulder), and turn lane detail. These can all be found in the Department's microstation "typical sections.cel" file. This file is available in the microstation set up at http://www.dot.state.ga.us/dot/preconstruction/adds/microstation/index.shtml.
23. See Chapter 11.6.1 and Figure 11.8 for a ramp typical section.

13.3 Paving Approval Checklist

The Paving Approval Checklist should accompany all pavement design submittals and is intended to provide guidance on developing cover sheet, typical sections, and pavement designs so that all design aspects are available to the Pavement Design Committee.

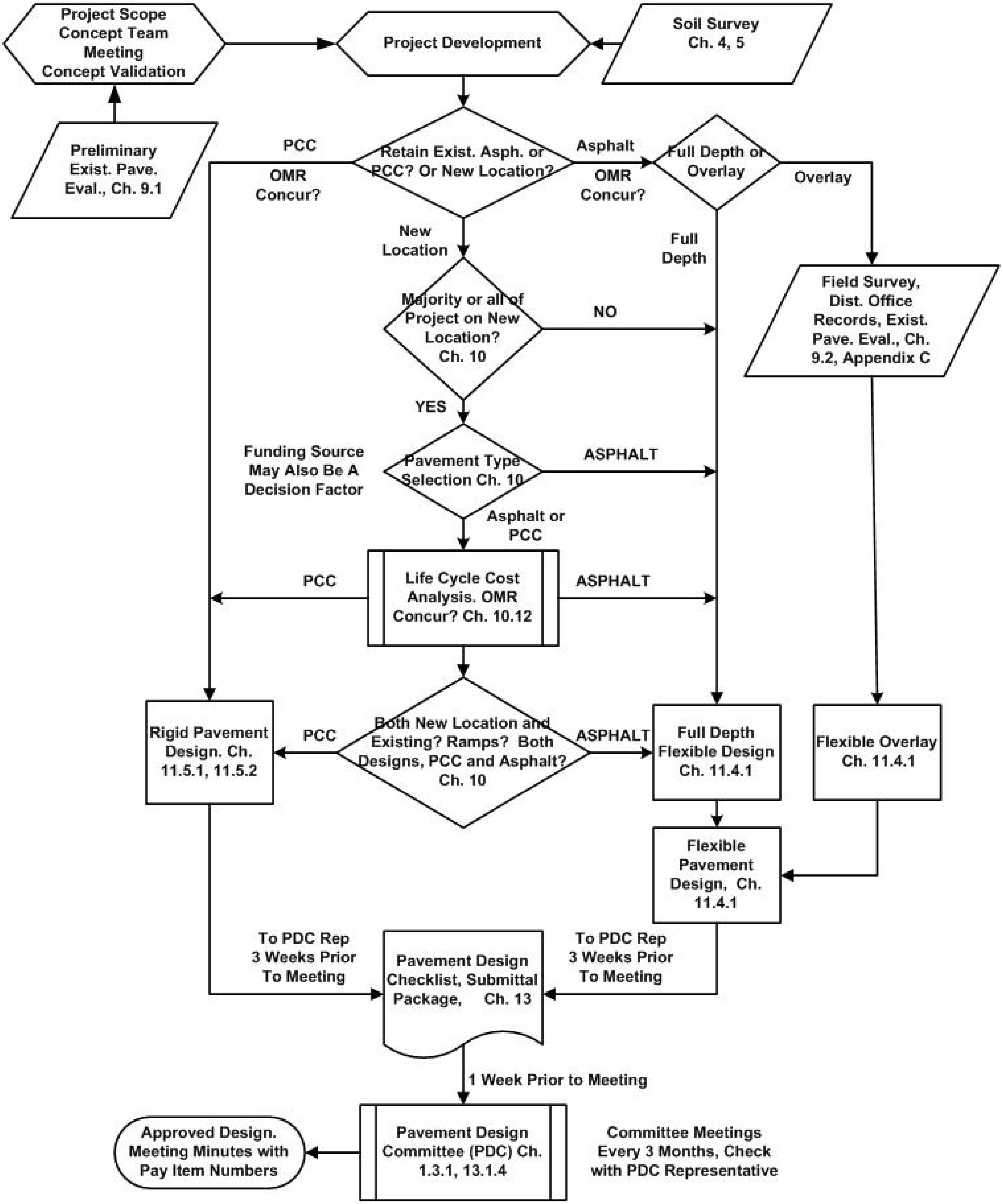
See **Pavement Approval Checklist, Figure 13.11.**:

13.4 Pavement Design Process

Pavement designs for most projects will be straight forward and may involve only ‘filling in the blanks’ both in the flexible and rigid design. Many projects will require intimate knowledge of a specific project including relation to adjoining projects, roadway history, and any future widening or traffic impacts.

- Figure 13.1 is a flow chart that tracks a pavement design thought process from initial concept to the approval process. Each decision point, process box, required document, or data box refers to a specific Chapter within the manual that will help with that particular point in the thought process. Experienced designers/engineers routinely go through these same steps, but for those that only design a pavement once or twice a year this flow chart may be a good reminder. It will not be surprising to see many designers/engineers copy only specific pages within the manual and create a smaller and specific pavement design manual. You are cautioned that the depth of knowledge, history, and guidance within this manual, though not needed on routine projects, will be of benefit on a unique project requiring more thoughtful consideration.
- Figures 13.2 through 13.10 are sample pavement designs. Each is discussed below relative to the thought process each requires.
- Figures 13.2 and 13.3, are rigid designs for I-20 and I-85 respectively. The Pavement Design Committee may require changes relative to the percent under/over design, and the location and usage of the proposed roadway.
- Flexible pavement design (asphalt) is required to be in the range of 10%-15% under design for rural projects. This allows for two asphalt overlays over the 20 year design period. An urban design (curb and gutter outside shoulder) requires 0%-5% under design. Only milling and inlay will be performed for routine maintenance so that the gutter spread is not adversely affected with future overlays.

- Figure 13.4, is a typical flexible design for a four-lane rural roadway (GDOT GRIP project). The designer/engineer should also be aware of the adjoining projects' pavement design, existing section, and proposed construction year. It may be beneficial to have the same pavement design on both projects, and each project's design criteria will serve as a check against each other. It's not uncommon that the soil support value, LDF, truck percentage, or ADT may differ from project to project, but if so, there should be a verification of the reasons why there are differences.
- Figure 13.5, is another typical flexible design for a four-lane rural roadway. The difference here is that the ADT is so low, the minimum design for US routes is used. Figure 13.6, is a flexible design for a four-lane urban (45 mph design, curb and gutter outside shoulder) roadway. This project has many side driveways and side roads that serve commercial businesses and strip shopping centers. Because there will be so many vehicles entering from various driveways, and trucks will also be making routine left turns, a lane distribution factor (LDF) of 0.6 is used instead of the normal 0.9 or 0.8. See discussion in Chapter 11.4.1. Also, this project will be widened to six lanes a few years after construction. The LDF for a six-lane urban roadway is recommended to be in the range of 0.6-0.8 (APD software). See also Appendix A.
- Figure 13.7, is a flexible design for a detour roadway. The difference here is that a Terminal Serviceability Index of 2.0 and design period of two years is used. Your design period may vary.
- Figures 13.8 through 13.10 represent alternate base designs that may be acceptable in specific counties. See Appendix I for areas of the State that are acceptable for the various base materials. Basically Graded Aggregate Base (GAB) is acceptable State wide and soil cement base is acceptable only in specific southern counties. Superpave asphalt base is not an acceptable alternate at this time for major projects (Section 13.1), but is for minor projects. The Soil Survey will recommend acceptable base materials that should be used in the pavement design analysis. If more than one base material is acceptable, then all must be submitted to the Pavement Design Committee, as well as summarized in construction plans and depicted separately in the typical sections as alternate designs.


Project Scope Concept Team Meeting Concept Validation
Project Development
Soil Survey Ch. 4, 5
Preliminary Exist. Pave. Eval., Ch. 9.1
PCC
OMR Concur?
Retain Exist. Asph. or PCC? Or New Location?
Asphalt
OMR Concur?
Full Depth or Overlay
Overlay
Full Depth
New Location
Majority or all of Project on New Location? Ch. 10
NO
YES
Field Survey, Dist. Office Records, Exist. Pave. Eval., Ch. 9.2, Appendix C
Funding Source May Also Be A Decision Factor
Pavement Type Selection Ch. 10
ASPHALT
Asphalt or PCC
PCC
Life Cycle Cost Analysis. OMR Concur? Ch. 10.12
ASPHALT
Rigid Pavement Design. Ch. 11.5.1, 11.5.2
PCC
Both New Location and Existing? Ramps? Both Designs, PCC and Asphalt? Ch. 10
ASPHALT
Full Depth Flexible Design Ch. 11.4.1
Flexible Overlay Ch. 11.4.1
Flexible Pavement Design, Ch. 11.4.1
To PDC Rep 3 Weeks Prior To Meeting
Pavement Design Checklist, Submittal Package, Ch. 13
To PDC Rep 3 Weeks Prior To Meeting
1 Week Prior to Meeting
Approved Design. Meeting Minutes with Pay Item Numbers
Pavement Design Committee (PDC) Ch. 1.3.1, 13.1.4
Committee Meetings Every 3 Months, Check with PDC Representative

10 G40 2/7/05

RIGID PAVEMENT DESIGN ANALSIS
(BASED ON AASHO INTERIM GUIDE FOR THE DESIGN OF RIGID PAVEMENT STRUCTURES)

P.I. NO. 0001795 PROJECT NUMBER NHS-0001-00 (795) COUNTY Richmond

:NGTH 2345" TYPE SECTION Full depth concrete main line and full depth asphalt shoulder

DESCRIPTION
Widening and reconstruction of I-20 (Cranes Creek Improvements)
Station 165 + 00 to Station 188 + 45

TYPE OF ADJOINING PAVEMENT: BEGINNING OF PROJECT: I-20 4 lane concrete w/asphalt shoulde
END OF PROJECT: Same

TRAFFIC DATA: 24 HR. TRUCK PERCENTAGE 17.3%
ONE-WAY AADT BEGINNING OF DESIGN PERIOD 0.52 x 71,600 = 37,232 VPD 2009 YEAR
ONE-WAY AADT END OF DESIGN PERIOD 0.52 x 116,620= 60,642 VPD 2029 YEAR
MEAN AADT (ONE WAY) 48,937 VPD

DESIGN LOADING:

DESIGN LANE TRAFFIC MEAN AADT		LDF		TRUCKS		18K ESAL		
48,937	X	60%	X	10.3% MU	X	2.68	=	8,105
48,937	X	60%	X	7.0% SU	X	0.5	=	1,028
48,937	X	60%	X	82.7% other	X	0.004	=	97

TOTAL DAILY LOADING = 9,230

TOTAL DESIGN PERIOD LOADING = 9,230 x 20 x 365 = 67,379,000

DESIGN DATA: SERVICEABILITY (Pt) 2.5 WORKING STRESS 450 psi

MODULUS OF SUBGRADE REACTION K = 190 pci
MODULUS OF SUBBASE REACTION K1 = K Subbase for 12" GAB + 3" Asphalt = 370 pci
RIAL DEPTH OF CONCRETE PAVEMENT E concrete = 3100000 psi
ACTUAL STRESS FROM NOMOGRAPH 550 psi PERCENT -UNDER DESIGN 22.2%

RECOMMENDED RIGID PAVEMENT STRUCTURE:

12" GAB, 3" asphalt & 12" concrete (1½" dowels)

3"- 19mm AC Superpave at Mix Design Level "A"

REMARKS: 12" GAB, 3" Asphalt & 13" Concrete (1½" dowels) --This section is 0% under-over designed.

PREPARED BY:

RECOMMENDED: STATE ROAD AND AIRPORT DESIGN ENGINEER DATE

APPROVED: STATE PAVEMENT ENGINEER DATE

FIGURE 13.2

RIGID PAVEMENT DESIGN ANALYSIS
(BASED ON AASHO INTERIM GUIDE FOR THE DESIGN OF RIGID PAVEMENT STRUCTURES)

P.I. NO.: 110620 **PROJECT NUMBER:** NH-IM-85-2(166) **COUNTY:** Barrow-Jackson
LENGTH: **TYPE SECTION: Full Depth Construction.**
DESCRIPTION: I-85 from SR 211 in Barrow County to SR 53 in Jackson County
TYPE OF ADJOINING PAVEMENT: **BEGINNING OF PROJECT:** MP 126 ±
END OF PROJECT: MP 129 ±

TRAFFIC DATA: 24 HR. TRUCK PERCENTAGE: 36% (32% MU, 4%SU)

ONE-WAY AADT BEGINNING OF DESIGN PERIOD:	26,300	**VPD**	2005	**YEAR**
ONE-WAY AADT END OF DESIGN PERIOD:	44,850	**VPD**	2025	**YEAR**
MEAN AADT (ONE WAY):	35,575	**VPD**		

DESIGN LOADING:
DESIGN LANE TRAFFIC

MEAN AADT		LDF		TRUCKS		18K ESAL		
35,575	X	80 %	X	32% MU	X	2.68	=	24,407
35,575	X	80 %	X	4 % Other	X	0.50	=	570
35,575	X	80 %	X	64% Other	X	0.004	=	73
						TOTAL DAILY LOADING	=	25,050

TOTAL DESIGN PERIOD LOADING = (25,050 loads/day)*(20 years)*(365 days/year) = 182,865,000 loads

DESIGN DATA: **SERVICEABILITY (Pt):** 2.5 **WORKING STRESS:** 450 psi
SOIL SUPPORT VALUE: 2.5
MODULUS OF SUBGRADE REACTION K = 130 pci
MODULUS OF SUBBASE REACTION K_1 = 220 pci on 12-in. GAB
MODULUS OF SUBBASE REACTION K_1 = 275 pci on 3-in. AC
TRIAL DEPTH OF CONCRETE PAVEMENT: 12"

ACTUAL STRESS FROM NOMOGRAPH: 765 psi
PERCENT OVER-UNDER DESIGN: 41 % underdesigned
PERCENT OVER-UNDER STRESS: 70 % overstressed

RECOMMENDED RIGID PAVEMENT STRUCTURE:

12 inches Plain Portland Cement Concrete with dowels
3 inches Asphalt Concrete Base
12 inches Graded Aggregate Base

REMARKS: 15.9 inches of PCC is required for 0% under/over design

PREPARED BY:

RECOMMENDED: ______________________________
STATE ROAD AND AIRPORT DESIGN ENGINEER **DATE**

APPROVED: ______________________________
STATE PAVEMENT ENGINEER **DATE**

FIGURE 13.3

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: EDS-19(64) **County:** Schley
P.I. no.: 322730
Description: US 19/SR 3 From SR 271 to SR 240

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 23.00%
AADT initial year of design period: 2,500 vpd (2006)
AADT final year of design period: 4,500 vpd (2026)
Mean AADT (one-way): 3,500 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
3,500	*	0.90	*	0.230	*	1.20	=	870

Total predicted design period loading = 870 * 20 * 365 = 6,351,000

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 3.00
Regional Factor: 1.60

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
12.5 mm Superpave	1.50	(38)	0.44	0.66
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.00	(25)	0.44	0.44
	3.00	(76)	0.30	0.90
Graded Aggregate Base	12.00	(305)	0.16	1.92

Required SN = 5.54 Proposed SN = 4.80

>>> Proposed pavement is 13.3% Underdesign <<<

Remarks: Full-Depth Section

Prepared by ______________________________
Date

Recommended ______________________________
State Consultant Design Engineer **Date**

Approved ______________________________
State Pavement Engineer **Date**

FIGURE 13.4

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: EDS-19(50) **County:** Sumter
P.I. no.: 322195
Description: SR 3/US 19 Widening & Reconstruction

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 12.00%
AADT initial year of design period: 5,000 vpd (2009)
AADT final year of design period: 7,500 vpd (2029)
Mean AADT (one-way): 6,250 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
6,250	*	0.90	*	0.120	*	1.20	=	811

Total predicted design period loading = 811 * 20 * 365 = 5,920,300

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 4.00
Regional Factor: 1.50

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
12.5 mm Superpave	1.50	(38)	0.44	0.66
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.00	(25)	0.44	0.44
	2.00	(51)	0.30	0.60
Graded Aggregate Base	10.00	(254)	0.16	1.60

Required SN = 4.84 Proposed SN = 4.18

>>> Proposed pavement is 13.6% Underdesign <<<

Remarks: New Design for EDS-19(50)

Prepared by ______________________________ **Date**

Recommended ______________________________
State Consultant Design Engineer **Date**

Approved ______________________________
State Pavement Engineer **Date**

FIGURE 13.5

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: STP-054-1(47) **County:** FORSYTH
P.I. no.: 122250
Description: SR 20 RECONSTRUCTION FROM FORSYTH CONN. TO SAMPLES ROAD

Traffic Data (NOTE: AADTs are one-way)

24-hour Truck Percentage: 6.00%
AADT initial year of design period: 20,850 vpd (2006)
AADT final year of design period: 33,250 vpd (2026)
Mean AADT (one-way): 27,050 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
27,050	*	0.60	*	0.060	*	1.06	=	1,033

Total predicted design period loading = 1033 * 20 * 365 = 7,540,900

Design Data

Terminal Serviceability Index: 2.50
Soil Support: 3.00
Regional Factor: 2.00

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
12.5 mm Superpave	1.50	(38)	0.44	0.66
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.00	(25)	0.44	0.44
	6.00	(152)	0.30	1.80
Graded Aggregate Base	12.00	(305)	0.16	1.92

Required SN = 5.85 Proposed SN = 5.70

>>> Proposed pavement is 2.5% Underdesign <<<

Remarks: SR 20 FULL DEPTH

Prepared by ______________________ **Date**

Recommended ______________________
State Consultant Design Engineer **Date**

Approved ______________________
State Pavement Engineer **Date**

Note new values! Recm as check against 0%-5% for Urban.

FIGURE 13.6

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: EDS-27(173) **County:** RANDOLPH/STEWART
P.I. no.: 422245
Description: US 27/SR 1 FROM CR 116 TO LUMPKIN BYPASS

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 16.00%
AADT initial year of design period: 1,250 vpd (2004)
AADT final year of design period: 2,250 vpd (2024)
Mean AADT (one-way): 1,750 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
1,750	*	0.90	*	0.160	*	1.17	=	296

Total predicted design period loading = 296 * 20 * 365 = 2,160,800

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 3.50
Regional Factor: 1.60

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
9.5 mm Superpave	1.25	(32)	0.44	0.55
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.25	(32)	0.44	0.55
	1.75	(44)	0.30	0.53
25 mm Superpave	5.00	(127)	0.30	1.50

Required SN = 4.48 Proposed SN = 4.01

>>> Proposed pavement is 10.6% Underdesign <<<

Remarks: Asphalt Base Alternate

Prepared by ________________________
Date

Recommended ________________________
State Consultant Design Engineer **Date**

Approved ________________________
State Pavement Engineer **Date**

FIGURE 13.10

10 GAV 2/7/05

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: NHS-0001-00(795) **County:** RICHMOND
P.I. no.: 0001795
Description: RECONSTRUCTION OF I-20, CRANES CREEK IMPROVEMENTS

Traffic Data (NOTE: AADTs are one-way)

24-hour Truck Percentage: 17.30%
AADT initial year of design period: 35,800 vpd (2009)
AADT final year of design period: 38,051 vpd (2011)
Mean AADT (one-way): 36,926 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
36,926	*	0.90	*	0.173	*	1.43	=	8,223

Total predicted design period loading = 8223 * 2 * 365 = 6,002,790

Design Data

Terminal Serviceability Index: 2.00
Soil Support: 4.00
Regional Factor: 1.60

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
12.5 mm OGFC	90 lb/sy	(50 kg/sm)	0.00	0.00
12.5 mm Superpave	1.50	(38)	0.44	0.66
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.00	(25)	0.44	0.44
	3.00	(76)	0.30	0.90
Graded Aggregate Base	10.00	(254)	0.16	1.60

Required SN = 4.57 Proposed SN = 4.48

>>> Proposed pavement is 2.0% Underdesign <<<

Prepared by ______________________ **Date**

Recommended ______________________ **Date**
State Consultant Design Engineer

Approved ______________________ **Date**
State Pavement Engineer

FIGURE 13.7

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: EDS-27(173) **County:** RANDOLPH/STEWART
P.I. no.: 422245
Description: US 27/SR 1 FROM CR 116 TO LUMPKIN BYPASS

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 16.00%
AADT initial year of design period: 1,250 vpd (2004)
AADT final year of design period: 2,250 vpd (2024)
Mean AADT (one-way): 1,750 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
1,750	*	0.90	*	0.160	*	1.17	=	296

Total predicted design period loading = 296 * 20 * 365 = 2,160,800

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 3.50
Regional Factor: 1.60

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
9.5 mm Superpave	1.25	(32)	0.44	0.55
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.25	(32)	0.44	0.55
	1.75	(44)	0.30	0.53
Soil-Cement Base	8.00	(203)	0.20	1.60

Required SN = 4.48 Proposed SN = 4.11

>>> Proposed pavement is 8.3% Underdesign <<<

Remarks: Soil Cement Base Alternate

Prepared by ______________________________ **Date**

Recommended ______________________________
State Consultant Design Engineer **Date**

Approved ______________________________
State Pavement Engineer **Date**

FIGURE 13.8

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: EDS-27(173) **County:** RANDOLPH/STEWART
P.I. no.: 422245
Description: US 27/SR 1 FROM CR 116 TO LUMPKIN BYPASS

Traffic Data (NOTE: AADTs are one-way)

24-hour Truck Percentage: 16.00%
AADT initial year of design period: 1,250 vpd (2004)
AADT final year of design period: 2,250 vpd (2024)
Mean AADT (one-way): 1,750 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
1,750	*	0.90	*	0.160	*	1.17	=	296

Total predicted design period loading = 296 * 20 * 365 = 2,160,800

Design Data

Terminal Serviceability Index: 2.50
Soil Support: 3.50
Regional Factor: 1.60

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
9.5 mm Superpave	1.25	(32)	0.44	0.55
19 mm Superpave	2.00	(51)	0.44	0.88
25 mm Superpave	1.25	(32)	0.44	0.55
	1.75	(44)	0.30	0.53
Graded Aggregate Base	10.00	(254)	0.16	1.60

Required SN = 4.48 Proposed SN = 4.11

>>> Proposed pavement is 8.3% Underdesign <<<

Remarks: Graded Aggregate Base Alternate

Prepared by ______________________________ **Date**

Recommended ______________________________
State Consultant Design Engineer **Date**

Approved ______________________________
State Pavement Engineer **Date**

FIGURE 13.9

Figure 13.11 **PAVING APPROVAL CHECKLIST**

PROJECT: _____ DATE: _____
P.I. NO.: _____ LET DATE: _____
COUNTY: _____
SUBMITTED BY:_____

☐ CONSULTANT PROJECT SCHEDULED COMPLETION DATE: _____

PLEASE CHECK ALL ITEMS BEFORE SUBMITTING YOUR DESIGN. UNDERSTANDABLY, SOME ITEMS ARE NOT APPLICABLE. **ENCLOSE THIS COMPLETED CHECKLIST WHEN SUBMISSION IS MADE.**

1) **GENERAL**

- ☐ COMPLETE COVER, TYPICAL SECTION, PAVING ANALYSIS, TRAFFIC DIAGRAM, EXISTING PAVING EVALUATION, SOIL SURVEY
- ☐ BRIEF DESCRIPTION OF PROJECT(I.E., PASSING LANE, BRIDGE REPL., ETC.) ____(ALSO ATTACH DETAILED PROJECT DESCRIPTION)

2) **COVER**

- ☐ PROJECT NUMBER
- ☐ COUNTY
- ☐ FEDERAL ROUTE
- ☐ STATE ROUTE
- ☐ P.I. NO.
- ☐ LENGTH
- ☐ % WITHIN COUNTY
- ☐ % WITHIN CONGRESSIONAL DISTRICT
- ☐ HORIZONTAL DATUM
- ☐ VERTICAL DATUM
- ☐ EAST OR WEST ZONE COORDINATES
- ☐ ENGLISH OR METRIC

DESIGN DATA

- ☐ TRAFFIC TWO WAY
- ☐ TRUCK % and 24 HOUR TRUCK %
- ☐ SPEED DESIGN
- ☐ FUNCTIONAL CLASSIFICATION
- ☐ FOS/EXEMPT/STATE FUNDED
- ☐ NORTH ARROW
- ☐ LOCATION SKETCH - FLAG PROJECT
- ☐ CHIEF ENGINEER
- ☐ OTHER
- ☐ PROJECT MIDPOINT
- ☐ PROJECT COOR. APPROX. MIDPOINT

3) PAVEMENT DESIGN FORM

- ☐ PROJECT NUMBER
- ☐ COUNTY
- ☐ P.I. NO.
- ☐ DESCRIPTION
- ☐ 24 HOUR TRUCK %
- ☐ AADT, ONE WAY
- ☐ LDF, LANE DISTRIBUTION FACTOR
- ☐ 18-K EQUIVALENT, ESAL
- ☐ TERMINAL SERVICEABILITY
- ☐ SOIL SUPPORT
- ☐ REGIONAL FACTOR
- ☐ PROPOSED PAVEMENT STRUCTURE
- ☐ % OVER/ UNDER DESIGN
- ☐ PRECONSTRUCTION ENGINEER'S REVIEW AND SIGNATURE
- ☐ DOES PLAN TYPICAL MATCH PAVING DESIGN ANALYSIS

4) TYPICAL

- ☐ MATERIAL LAYERS LABELED
- ☐ ALLOWABLE RANGE TABLE
- ☐ GUARDRAIL DETAIL
- ☐ SLOPE CONTROLS
- ☐ CROSS-SLOPE INDICATED
- ☐ INDENTATION RUMBLE STRIPS
- ☐ SPREAD RATES
- ☐ ALTERNATE SECTIONS IF REQUIRED
- ☐ CLASS “B” CONCRETE DETAIL
- ☐ SUPERPAVE MIX DESIGN LEVEL NOTE
- ☐ PAVEMENT REINFORCING FABRIC DETAIL

REMEMBER

- ☐ DESIGN FOR CLEAR ZONE
- ☐ IF DESIGN VARIANCES ARE USED, PLEASE PROVIDE ADEQUATE JUSTIFICATION
- ☐ NEW PAVEMENT – UNDER DESIGN - 10-15% RURAL, 0-5% URBAN AND BRIDGE APPROACHES
- ☐ SUBMIT DESIGN AROUND 6 TO 9 MONTHS PRIOR TO LET DATE OR CONSULTANT COMPLETION DATE

14. Pavement Glossary of Terms

A B C D E F G H I J K
L M N O P R S T U W

A Top

AASHTO – American Association of State Highway and Transportation Officials

Acceleration – Increase in rate of hardening or strength development of concrete.

Accelerator – An admixture which, when added to concrete, mortar, or grout, increases the rate of hydration of hydraulic cement, shortens the time of set, or increases the rate of hardening or strength development.

ACPA – American Concrete Pavement Association

ADT – Average Daily Traffic. *In pavement design ADT influences pavement design. The higher* the ADT is, the thicker the required pavement will be. Plan cover sheets provide is two-way ADT. Pavement design requires one-way ADT.

AADT – Annual Average Daily Traffic.

ADTT – Annualized Daily Truck Traffic.

Adhesion Loss – The loss of bond between a joint sealant material and the concrete joint face noted by physical separation of the sealant from either or both joint faces.

Adhesives – The group of materials used to join or bond similar or dissimilar materials; for example, in concrete work, the epoxy resins.

Agency Costs – See Annual Costs.

Aggregate – Granular material, such as sand, gravel, crushed stone, crushed hydraulic cement concrete, or iron blast furnace slag, used with a hydraulic cementing medium to produce either concrete or mortar.

Aggregate Interlock – The projection of aggregate particles or portion of aggregate particles from one side of a joint or crack in concrete into recesses in the other side of the joint or crack so as to effect load transfer in compression and shear and maintain mutual alignment

Alkali-Silica Reaction – The reaction between the alkalies (sodium and potassium) in Portland cement binder and certain siliceous rocks or minerals, such as opaline chert, strained quartz, and acisic volcanic glass, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.

Alligator Cracking – A series of interconnecting cracks in an asphalt pavement surface forming a pattern that resembles an alligator's hide or chicken wire. In its early stages, alligator cracking may be characterized by a single longitudinal crack in the wheel path. The cracks indicate fatigue failure of the surface layer generally caused by repeated traffic loadings. Hence, the term fatigue cracking is also used.

Analysis Period – The period of time used in making economic comparisons between rehabilitation alternatives. In other words the period of time for which a life cycle cost analysis is to be made; it must include at least one rehabilitation activity in order to realize the full benefit of the initial investment. The analysis period should not be confused with the pavement's design life (performance period).

Annual Costs – Any costs associated with the annual maintenance and repair of the facility.

APD72 – Is a software program that was developed internally at GDOT to automate the solution of the 1972 Flexible Pavement Design Nomographs.

Asphalt **–** A brown to black bituminous substance that is chiefly obtained as a residue of petroleum refining and that consists mostly of hydrocarbons.

Asphalt Concrete Base - A base type that utilizes hot mix asphalt concrete placed directly on subgrades of high soil support values. This is a common base material in south Georgia.

ASR – See, Alkali-Silica Reaction

ASTM – American Society for Testing and Materials

Asphalt Tack Coat – A light application of asphalt, usually asphalt emulsion diluted with water. It is used to ensure a bond between two bituminous pavement layers.

Asset Management – A systematic process of maintaining, upgrading, and operating physical assets cost-effectively. It combines engineering principles with sound business practices and economic theory, and it provides tools to facilitate a more organized, logical approach to decision-making. Thus, asset management provides a frame work for handling both short and long-range planning.

Axle Load – The portion of the gross weight of a vehicle transmitted to a structure or a pavement through wheels supporting a given axle.

B **Top**

Backer Material – A compressible material that is placed in joints or cracks of rigid pavements, before applying a sealant material to prevent bonding of the sealant on the bottom of the joint, control sealant depth, and prevent sagging of the sealant.

Bar Spacing - The distance between parallel reinforcing bars, measured center to center of the bars perpendicular to their longitudinal axis.

Bar Support - A rigid device used to support or hold reinforcing bars in proper position to prevent displacement before or during concrete placing.

Bedrock or Ledge Rock - In-place rock; or rock in its native location still attached to the parent formation.

Bench Gravels - Gravel beds on the side of a valley above the present stream level. Represents part of a stream bed when it was at a higher level.

Bitumen - Any of various mixtures of hydrocarbons (as tar) often together with their non-metallic derivatives that occur naturally or are obtained as residues after heat-refining petroleum

Bituminous - Resembling, containing or impregnated with bitumen.

Backer Rod - Foam cord that inserts into a joint sealant reservoir and is used to shape a liquid joint sealant and prevent sealant from adhering to or flowing out of the bottom of the reservoir.

Bar Chair - An individual supporting device used to support or hold reinforcing bars in proper position to prevent displacement before or during concreting.

Bar Spacing - The distance between parallel reinforcing bars, measured center to center of the bars perpendicular to their longitudinal axis.

Bar Support - A rigid device used to support or hold reinforcing bars in proper position to prevent displacement before or during concrete placing.

Base – According to GDOT specifications a base is one or more layers of specified material of design thickness placed on the subgrade or subbase to support a surface course.

Bituminous Pavement – A pavement comprising an upper layer or layers of aggregate mixed with a bituminous binder, such as asphalt, coal tars, and natural tars for purposes of this terminology; surface treatments such as chip seals, slurry seals, sand seals, and cape seals are also included.

Bleeding – Excess asphalt binder occurring on the pavement surface. The bleeding may create a shiny, glass-like surface that may be tacky to the touch. Bleeding is usually found in the wheel paths.

Blinding - The condition in which soil particles block the voids at the surface of a geotextile, therefore reducing hydraulic conductivity of the geotextile, a formation of surface crust or cake.

Block Cracking – A rectangular pattern of cracking in asphalt pavements that is caused by hardening and shrinkage of the asphalt. Block cracking typically occurs at a uniformly spaced interval.

Blow-up – Buckling and shattering of PCC pavement resulting from thermal expansion and the resultant compressive forces exceeding the strength of the material.

Bond Breaker – A material used to prevent adhesion of newly placed concrete from other material, such as a substrate. Any material used to prevent bonding or to separate adjacent pavement layers. Thin bituminous layers are often used as bond breaker layers between a concrete pavement and an unbonded concrete overlay.

Bonded Concrete Overlay – Thin layer of new concrete (2-4 inches) placed onto slightly deteriorated existing concrete pavement with steps taken to prepare old surface to promote adherence of new concrete.

Increase in the pavement structure of a concrete pavement by addition of concrete thickness in direct contact with and adhering to the existing concrete surface. This method is used to correct either functional or structural deficiencies. This is not a standard GDOT rehabilitation method.

Burlap - A coarse fabric of jute, hemp, or less commonly flax, for use as a water-retaining cover for curing concrete surfaces; also called Hessian.

Butt Joint - A plain, square joint between two concrete slabs.

C Top

California Bearing Ratio (CBR) - The ratio of the force per unit area required to penetrate a soil mass with a 19.4 sq cm circular piston at the rate of 1.27 mm per min to the force required for corresponding penetration of a standard crushed-rock base material; the ratio is usually determined at 2.5 mm penetration.

Carbide Milling – Surface removal or sawing done with a carbide milling machine. Machine uses a blade or arbor equipped with carbide-tipped teeth that impact and chip concrete or asphalt.

Chemically Curing Sealant – A material that reaches its final properties through the reaction of the component materials when mixed.

Chip Seal – A surface treatment in which the pavement is sprayed with asphalt (generally emulsified) and then immediately covered with aggregate and rolled. Chip seals are used primarily to seal the surface of a pavement with non load-associated cracks and to improve surface friction, although they also are commonly used as a wearing course on low volume roads.

Coefficient of Thermal Expansion - Change in linear dimension per unit length or change in volume per unit volume per degree of temperature change.

Compaction - The process whereby the volume of freshly placed mortar or concrete is reduced to the minimum practical space, usually by vibration, centrifugation, tamping, or some combination of these; to mold it within forms or molds and around embedded parts and reinforcement, and to eliminate voids other than entrained air. See also Consolidation

Compressible Insert - Board used to separate a partial-depth patch from an adjacent slab, usually consisting of a 12-mm thick Styrofoam or compressed fiber material that is impregnated with asphalt.

Compressive Strength - The measured resistance of a concrete or mortar specimen to axial loading; expressed as pounds per square inch (psi) of cross-sectional area.

Concrete - A composite material that consists essentially of a binding medium in which is embedded particles or fragments of relatively inert material filler. In Portland cement concrete, the binder is a mixture of Portland cement and water; the filler may be any of a wide variety of natural or artificial aggregates.

Consistency - The relative ease with which a cohesive soil can be deformed. It is usually expressed qualitatively by terms such as very soft, soft, medium stiff, stiff, hard and very hard.

Consolidated Drained Test (Slow Test) - A soil test in which essentially complete consolidation under the confining pressure is followed by additional axial (or shearing) stress applied in such a manner that even a fully saturated soil of low permeability can adapt itself completely (fully consolidate) to the changes in stress due to the additional axial (or shearing) stress.

Consolidated Undrained Test (Consolidated Quick Test) - A test in which complete consolidation under the vertical load (in a direct shear test) or under the confining pressure (in a triaxial test) is followed by a shear at constant water content.

Consolidation - The process of inducing a closer arrangement of the solid particles in freshly mixed concrete or mortar during placement by the reduction of voids, usually by vibration, centrifugation, tamping, or some combination of these actions; also applicable to similar manipulation of other cementitious mixtures, soils, aggregates, or the like. See also Compaction.

Consolidation Initial (Initial Compression) – A comparatively sudden reduction in volume of a soil mass under an applied load due principally to expulsion and compression of gas in the soil voids preceding primary consolidation.

Consolidation Test Primary (Primary Compression) - The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids.

Consolidation, **Secondary**- The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

Consolidation Test - A test where the specimen is laterally confined in a ring and is compressed between porous plates.

Consolidation - Time Curve (Consolidation Curve) - A curve that shows the relation between(1) the degree of consolidation and (2) the elapsed time after the application of a given increment of load.

Construction Joint - The junction of two successive placements of concrete, typically with a keyway or reinforcement across the joint.

Continuously Reinforced Pavement - A pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints.

Contract - Decrease in length or volume. See also Expand, Shrinkage, Swelling, and Volume Change.

Contraction Joint - A plane, usually vertical, separating concrete in a structure of pavement, at a designated location such as to prevent formation of objectionable shrinkage cracks elsewhere in the concrete. Reinforcing steel is discontinuous.

Control Joint - See Contraction Joint.

Corner Break - A portion of the slab separated by a crack that intersects the adjacent transverse or longitudinal joints at about a 45º angle with the direction of traffic. The length of the sides is usually from 0.3 meters to one-half of the slab width on each side of the crack.

Course - In concrete construction, a horizontal layer of concrete, usually one of several making up a lift; in masonry construction, a horizontal layer of block or brick. See also Lift.

Cover - In reinforced concrete, the least distance between the surface of the reinforcement and the outer surface of the concrete.

Cohesion – The internal bond within a joint sealant material. Cohesion loss is seen as a noticeable tear along the surface and through the depth of the sealant.

Cold Applied Sealant – A crack-sealing compound that is applied in an unheated state (generally at ambient temperature) and then reaches final properties through a curing process.

Cold Milling – A process of removing pavement material from the surface of the pavement either to prepare the surface to receive overlays (by removing rutting, and surface irregularities) or to restore pavement cross slopes and profile. Also used to remove oxidized asphalt concrete. See also Carbide Milling.

Compressible Insert – Material used to separate freshly placed concrete (such as from a partial-depth or full-depth repair) from existing hardened concrete. This usually consists of a 12-mm (0.5 in) thick Styrofoam or compressed fiber material that is impregnated with asphalt.

Concrete – See Portland Cement Concrete.

Construction Joint – A joint constructed in a transverse direction in PCC pavements to control cracking of the slab as it cures. Highway construction joints are created by sawing the concrete. GDOT's typical joint spacing is 15 feet for Interstate highways, and 20 feet for non interstates.

Continuously Reinforced Concrete Pavement (CRCP) – PCC pavement constructed with sufficient longitudinal steel reinforcement to control transverse crack spacings and openings in lieu of transverse contraction joints for accommodating concrete volume changes and load transfer.

Corner Break – A portion of a concrete slab separated by a crack that intersects the adjacent transverse or longitudinal joints at about a 45 degree angle with the direction of traffic. The length of the sides is usually from 0.3 meters (1 ft) to one-half of the slab width on each side of the crack.

Corrective Maintenance – Maintenance performed once a deficiency occurs in the pavement; for example, pothole filling, or spall repair.

CPR (Concrete Pavement Restoration) – A series of repair techniques used to preserve or improve the structural capacity or functional characteristics of a PCC pavement. CPR techniques each have a unique purpose to repair or replace a particular distress (kind of deterioration) found in PCC pavement and to manage the rate of deterioration. CPR techniques include:

- Full-depth repair
- Partial-depth repair
- Diamond grinding
- Joint and crack resealing
- Slab stabilization
- Dowel Bar Retrofit
- Cross-stitching cracks or longitudinal joints
- Retrofitting concrete shoulders
- Retrofitting edge drains

CRC Pavement (CRCP) - Continuously reinforced concrete pavement. See Continuously Reinforced Pavement.

Crack – Fissure or discontinuity of the pavement surface not necessarily extending through the entire thickness of the pavement. Cracks generally develop after initial construction of the pavement and may be caused by thermal effects, excess loadings, or excess deflections.

Cracking – The process of contraction or the reflection of stress in the pavement.

Crack Filling – The placement of materials into non-working cracks to substantially reduce the intrusion of incompressibles and infiltration of water, while also reinforcing the adjacent pavement. Crack filling should be distinguished from crack sealing.

Crack Sealing – A maintenance procedure that involves placement of specialized materials into working cracks using unique configurations to reduce the intrusion of incompressibles into the crack and to prevent infiltration of water into the underlying pavement layers. See also Working Crack.

Cross Stitching – A repair method that involves the drilling of holes diagonally across a crack in PCC pavement into which steel reinforcement bars are inserted and epoxied in place. The holes are alternated from side to side of the crack on a pre-determined spacing. This technique is generally used for longitudinal cracks that are still in no worse than fair condition. Cross-stitching increases slab integrity by adding steel reinforcement to hold the crack together.

Cure – A period of time following placement and finishing of a material such as concrete during which desirable engineering properties (such as strength) develop. Improved properties may be achieved by controlling temperature or humidity during curing.

Curing – Maintenance of a satisfactory moisture content and temperature in concrete during its early stages following placing and finishing to ensure proper hydration of the cement and proper hardening of the concrete.

Curing Blanket – A built-up covering of sacks, matting, Hessian, straw, waterproof paper, or other suitable material placed over freshly finished concrete. See also Burlap.

Curing Compound – A liquid that can be applied as a coating to the surface of newly placed concrete to retard the loss of water or, in the case of pigmented compounds, also to reflect heat so as to provide an opportunity for the concrete to develop its properties in a favorable temperature and moisture environment. See also Curing.

D

[**Top**]

DHV - See Design Hourly Volume.

Daylight Refers to drainage (see below); a process that allows water to flow out of the subbase / base into ditches, instead of using pipes and sophisticated drainage systems.

Dense-Graded Asphalt Pavement – An overlay or surface course consisting of a mixture of asphalt binder and a well-graded (also called dense-graded) aggregate. A well-graded aggregate is uniformly distributed throughout a full range of sieve sizes. See also Hot Mix Asphalt.

Depression – Localized pavement surface areas at a lower elevation than the adjacent paved areas.

Design Hourly Volume (DHV) - The traffic volume expected to be used by a highway segment during the 30th highest hour of the design year. The Design Hour Volume (DHV) is related to AADT by the K-Factor.

Design Life – The expected life of a pavement from its opening to traffic until structural rehabilitation is needed. The typical reporting of pavement design life does not include the life of the pavement with the application of preventive maintenance. See also Analysis Period and Performance Period.

Deterioration 1) Physical manifestation of failure (for example, cracking de-lamination, flaking, pitting, scaling, spalling, staining) caused by environmental or internal autogenous influences on rock and hardened concrete as well as other materials; 2) decomposition of material during either testing or exposure to service.

Diamond Grinding – A process that uses a series of diamond-tipped saw blades mounted on a shaft or arbor to shave the upper surface of a pavement to remove bumps, restore pavement rideability, and improve surface friction. See also CPR.

Discount Rate – The rate of interest reflecting the investor's time value of money used to determine discount factors for converting benefits and costs occurring at different times to a baseline date. Discount rates can incorporate an inflation rate depending on whether real discount rates or nominal discount rates are used. The discount rate is often approximated as the difference between the interest rate and the inflation rate.

Distress Physical manifestation of deterioration and distortion in a concrete structure as the result of stress, chemical action, and/or physical action.

Dowel – 1) A load transfer device. Most commonly a plain round steel bar which extends into two adjoining slabs of a PCC pavement at a transverse joint placed parallel to the center line so as to transfer shear loads. 2) A deformed reinforcing bar intended to transmit tension, compression, or shear through a construction joint.

Dowel Bar (Dowelbar) -See Dowel.

Dowel Basket See Load-Transfer Assembly.

Dowel Bar Retrofit (DBR) – A rehabilitation technique that is used to increase the load transfer capability of existing jointed PCC pavements by placement of dowel bars across joints and/or cracks that exhibit poor load transfer. See also CPR.

Drainage The interception and removal of water from, on, or under an area or roadway; the process of removing surplus ground or surface water artificially; a general term for gravity flow of liquids in conduits.

E

Top

Early Strength Strength of concrete developed soon after placement, usually during the first 72 hours.

Econocrete Portland cement concrete designed for a specific application and environment and, in general, making use of local commercially produced aggregates. These aggregates do not necessarily meet conventional quality standards for aggregates used in pavements.

18-k Equivalent - a conversion of particular truck configurations associated with the specific road to an 18-k equivalent loading. The proposed pavement is theoretically analyzed and the number of equivalent loading is Interstate generally carry more MU(multi-units) and fewer SU single units. Local road or vice versa.

- 10 MU x 1.5(18-k equiv) = 15 loadings
- 10 SU x .40(18-k equiv) = 4 loadings

Emulsified Asphalt – A liquid mixture of asphalt binder, water, and an emulsifying agent. Minute globules of asphalt are suspended in water by using an emulsifying agent. These asphalt globules are either anionic (negatively charged) or cationic (positively charged).

Equivalent Single Axle Loads (ESAL's) Summation of equivalent 18,000-pound single axle loads used to combine mixed traffic to design traffic for the design period.

Equivalent Uniform Annual Cost (EUAC) – The net present value of all discounted cost and benefits of an alternative as if they were to occur uniformly throughout the analysis period. Net Present Value (NPV) is the discounted monetary value of expected benefits, such as benefits minus costs.

Expansion Increase in length or volume. See also Autogenous Volume Change, Contraction, Moisture Movement, Shrinkage, and Volume Change.

Expansion Joint See Isolation Joint.

F Top

Fatigue Cracking – See Alligator Cracking.

Faulting – Differential vertical displacement of a slab or other member adjacent to a joint or crack. Faulting commonly occurs at transverse joints of PCC pavements that do not have adequate load transfer.

FHWA - Federal Highway Administration.

Fiber Modified Sealant – Generally a hot-applied sealant that is composed of unmodified or modified asphalt cement and heat resistant polymeric fibers and is used for sealing cracks in asphalt concrete pavements.

Flexible Pavement - A pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability; cementing agents, where used, are generally bituminous (asphaltic) materials as contrasted to Portland cement in the case of rigid pavement. See also Rigid Pavement.

Flexural Strength - A property of a material or structural member that indicates its ability to resist failure in bending. See also Modulus of Rupture.

Fly Ash The finely divided residue resulting from the combustion of ground or powdered coal and which is transported from the fire box through the boiler by flu gasses; Used as mineral admixture in concrete mixtures.

Fog Seal – A light application of slow setting asphalt emulsion diluted with water and without the addition of any aggregate applied to the surface of a bituminous pavement. Fog seals are used to renew aged asphalt surfaces, seal small cracks and surface voids, or adjust the quality of binder in newly applied chip seals.

Form - A temporary structure or mold for the support of concrete while it is setting and gaining sufficient strength to be self-supporting.

Free Edge – An unrestrained pavement boundary.

Full-depth Patching 1) Removing and replacing at least a portion of a concrete slab to the bottom of the concrete, in order to restore areas of deterioration. 2) Removal and replacement of a segment of a flexible pavement to the level of the subgrade in order to restore areas of deterioration.

Functional Performance – A pavement's ability to provide a safe, smooth riding surface. These attributes are typically measured in terms of ride quality (see International Roughness Index) or skid resistance.

G

Top

Graded Aggregate Base (GAB) A type of base that utilizes processed crushed stone or graded aggregate exclusively. This type of base is exclusively used in areas of low soil support values, specifically in north Georgia.

Geotextiles A geotextile is a synthetic permeable textile manufactured from man made fabrics. Within the context of pavement design, geotextiles are intended to have beneficial engineering properties such as limiting the intrusion of fines from the subgrade or assist in strengthening the subgrade.

Grinding Head – Arbor or shaft containing numerous diamond blades or carbide teeth on diamond grinding or cold milling equipment.

Grooving – The process used to cut slots into a pavement surface (usually, although not always, PCC) to provide channels for water to escape beneath tires, improving skid resistance and reducing the potential for hydroplaning.

Grout A mixture of cementitious material and water, with or without aggregate, proportioned to produce a pourable consistency without segregation of the constituents; also, a mixture of other composition but of similar consistency. See also Sand Grout.

H

Top

Hairline Cracking Barely visible cracks in random pattern in an exposed concrete surface which do not extend to the full depth or thickness of the concrete, and which are due primarily to drying shrinkage.

Hardening When Portland cement is mixed with enough water to form a paste, the compounds of the cement react with water to form cementitious products that adhere to each other and to the intermixed sand and stone particles and become very hard. As long as moisture is present, the reaction may continue for years, adding continually to the strength of the mixture.

Heater Scarification – The initial phase of a hot in-place recycling (HIR) process in which the surface of the old pavement is heated and mechanically raked before being removed and recycled.

HMAC - Hot Mix Asphalt Concrete; asphalt pavement

Hot Air Lance – A device that uses heated compressed air to clean, dry, and warm cracks prior to sealing.

Hot-pour or hot applied Sealant - Joint sealing materials that require heating for installation, usually consisting of a base of asphalt or coal tar. It is applied in a molten state and cures primarily by cooling to ambient temperature.

Hot In-Place Recycling (HIR) – A process which consists of softening the existing asphalt surface with heat, mechanically removing the surface material, mixing the material with a recycling agent, adding virgin asphalt and aggregate to the material (if required), and then replacing the material on the pavement.

Hot Mix Asphalt Concrete (HMAC or HMA) – A thoroughly controlled mixture of asphalt binder and well-graded, high quality aggregate thoroughly compacted into a uniform dense mass. HMAC pavements may also contain additives such as anti-stripping agents and polymers.

Hydrated Lime A dry powder obtained by treating quicklime with sufficient water to convert it to calcium hydroxide.

Hydration The chemical reaction between cement and water which causes concrete to harden.

Hydraulic Cement A cement that is capable of setting and hardening under water due to the chemical interaction of the water and the constituents of the cement.

Hydroplaning – Loss of contact between vehicle tires and roadway surface that occurs when vehicles travel at high speeds on pavement surfaces with standing water.

I **Top**

Inlay 1) A form of reconstruction where new concrete is placed into an area of removed pavement; Removal may be an individual lane, all lanes between the shoulders or only partly through a slab. 2) A form of reconstruction where new asphalt pavement is placed into an area of milled pavement. The removal may be in an individual lane, all lanes between the shoulders or only partly through a full depth asphalt pavement.

Initial Costs – All costs associated with the initial design and construction of a facility, placement of a treatment, or any other activity with a cost component.

International Roughness Index (IRI) – A measure of a pavement's longitudinal surface profile as measured in the wheelpath by a vehicle traveling at typical operating speeds. It is calculated as the ratio of the accumulated suspension motion to the distance traveled obtained from a mathematical model of a standard quarter car traversing a measured profile at a speed of 80 km/h (50 mph). The IRI is expressed in units of meters per kilometer (inches per mile) and is a representation of pavement roughness.

Isolation Joint A pavement joint that allows relative movement in three directions and avoids formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted. large closure movement to prevent development of lateral compression between adjacent concrete slabs; usually used to isolate a bridge.

J Top

Joint – 1) A pavement discontinuity made necessary by design or by interruption of a paving operation. 2) A plane of weakness to control contraction cracking in concrete pavements. A joint can be initiated in plastic concrete or green concrete and shaped with later process.

Joint Depth The measurement of a saw cut from the top of the pavement / slab surface to the bottom of the cut.

Joint Deterioration See Spalling.

Joint Filler Compressible material used to fill a joint to prevent the infiltration of debris and to provide support for sealant.

Joint, Construction See Construction Joint.

Joint, Contraction See Contraction Joint.

Joint, Expansion See Expansion Joint.

Joint Filler – Compressible material used to fill a joint to prevent the infiltration of debris.

Joint Sealant – Compressible material used to minimize water and solid debris infiltration into the sealant reservoir and joint.

Joint Seal Deterioration - Break down of a joint or crack sealant, such as by adhesion or cohesion loss, which contributes to the failure of the sealant system. Joint seal deterioration permits incompressible materials or water to infiltrate into the pavement system.

Joint Shape Factor – Ratio of the vertical to horizontal dimension of the joint sealant reservoir. Factor can vary depending on type of sealant specified.

Jointed Plain Concrete Pavement (JPCP) – PCC pavement constructed with regularly spaced transverse joints to control all natural cracks expected in the concrete. Dowel bars may be used to enhance load transfer at transverse contraction joints (depending upon the expected traffic); however, there is no mid-slab temperature reinforcement.

K Top

Keyway A recess or groove in one lift or placement of concrete which is filled with concrete of the next lift, giving shear strength to the joint.

L Top

Lane Distribution Factor (LDF) - Lane Distribution Factor - Typically, the outside lane will carry the highest percentage of truck traffic. The lane with the heaviest amount of truck traffic will be the lane we design for typically, 2-lane LDF - 100%, 4-lane LDF - 80 to 90%, 6-lane LDF 70%

Layer coefficient – A measure of the relatve ability of a unit thickness of a given material to function as a structural component of the pavement.

Life Cycle Costing – An economic assessment of an item, system, or facility and competing design alternatives considering all significant costs of ownership over the economic life, expressed in terms of equivalent dollars.

Life-Cycle Cost Analysis - The process used to compare projects based on their initial cost, future cost and salvage value, which accounts for the time value of money.

Life Extension – The extension of the performance period of the pavement through the application of preventive pavement treatments.

Lift - The concrete placed between two consecutive horizontal construction joints, usually consisting of several layers or courses.

Load-Transfer Assembly - Most commonly, the basket or carriage designed to support or link dowel bars during concreting operations so as to hold them in place, in the desired alignment.

Load Transfer Device - See Dowel.

Load Transfer Efficiency – A measure of the ability of a joint or crack to transfer a portion of a load applied on one side of a joint or crack to the other side of the joint or crack.

Load Transfer Restoration (LTR) See Retrofit Dowel Bars.

Longitudinal Crack – A crack or discontinuity in a pavement that runs generally parallel to the pavement centerline. Longitudinal cracks may occur as a result of poorly constructed paving lane joints, thermal shrinkage, inadequate support, reflection from underlying layers, or as a precursor to fatigue cracking.

Note: Longitudinal cracking that occurs in the wheel path is generally indicative of alligator cracking.

Longitudinal Joint – A constructed joint in a pavement layer that is oriented parallel to the pavement centerline.

Longitudinal Reinforcement - Reinforcement essentially parallel to the long axis of a concrete member or pavement.

M

Top

Microsurfacing – A mixture of polymer modified asphalt emulsion, mineral aggregate, mineral filler, water, and other additives, properly proportioned, mixed, and spread on a paved surface. Microsurfacing differs from slurry seal in that it can be used on high volume roadways to correct wheel path rutting and provide a skid resistant pavement surface.

Mineral Filler – A finely divided mineral product with at least 70% passing the No. 200 sieve. Commonly used mineral fillers include, limestone dust, hydrated lime, Portland cement, and fly ash.

Minimum Application Temperature – The minimum temperature, as recommended by the manufacturer, to which a hot-applied sealant for pavement cracks or joints must be heated while conforming to all specification requirements and result in appropriate application characteristics.

Modified Asphalt Chip Seal – A variation on conventional chip seals in which the asphalt binder is modified with a blend of ground tire or latex rubber, or polymer modifiers to enhance the elasticity and adhesion characteristics of the binder.

Modulus of Rupture - A measure of the ultimate load-carrying capacity of a beam, sometimes referred to as "rupture modulus" or "rupture strength." It is calculated for apparent tensile stress in the extreme fiber of a transverse test specimen under the load that produces rupture. See also Flexural Strength.

Multi Unit Trucks (MU) - Multi Unit Trucks are trucks with three or more axles. According to the FHWA Classification scheme this comprises of vehicles from Class 6 through Class 13.

N **Top**

NCHRP - National Cooperative Highway Research Program

Net Present Value – The value of future expenditures or costs discounted to today's dollars using an appropriate discount rate.

NHI - National Highway Institute

O **Top**

Off-System Roads - Roads that are not owned or maintained by GDOT. Local roads such as County roads fall in this category.

On-System Roads - Roads that are owned and maintained by GDOT.

Open-Graded Friction Course (OGFC) – A thin HMA surface course consisting of a mix of an asphalt binder and open-graded (also called uniformly graded) aggregate. An OGFC helps to eliminate standing water on a pavement surface, which reduces tire spray and hydroplaning potential. It has no structural value in pavement design computations.

Overbanding – Overfilling of a joint or crack reservoir so that a thin layer of crack or joint sealant is spread onto the pavement surface center over the joint or crack.

Overlay - The addition of a new material layer onto an existing pavement surface. See also Resurfacing.

Overlay, Bonded - See Bonded Concrete Overlay.

Overlay, Unbonded - See Unbonded Concrete Overlay.

Overlay, UTW - See Ultra-thin Whitetopping.

Overlay, Whitetopping -See Whitetopping.

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Partial-Depth Patching – Repairs of localized areas of surface deterioration of PCC pavements, usually for compression spalling problems, severe scaling, or other surface problems that are within the upper one-third of the slab depth.

Patch – Placement of a repair material to replace a localized defect in the pavement surface.

Pavement Distress – External (visible) indications of pavement defects or deterioration.

Pavement Preservation – The sum of all activities undertaken to provide and maintain serviceable roadways. This includes corrective maintenance and preventive maintenance, as well as minor rehabilitation projects.

Pavement Preventive Maintenance – Planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without increasing the structural capacity).

Pavement Reconstruction – Replacement of an existing pavement structure by the placement of the equivalent of a new pavement structure. Reconstruction usually involves complete removal and replacement of the existing pavement structure and may include new and/or recycled materials.

Pavement Rehabilitation – Structural enhancements that extend the service life of an existing pavement and/or improve its load carrying capability. Rehabilitation techniques include restoration treatments and structural overlays.

Performance Period – The period of time that an initially constructed or rehabilitated pavement structure will perform before reaching its terminal serviceability.

Plant Mix – See Hot Mix Asphalt.

Point Bearing – Concentration of compressive stressed between small areas. May occur when a partial-depth patch in Portland cement concrete pavement is made without the compressible insert. Also, slab expansion in hot weather forces an adjacent slab to bear directly against a small partial-dept patch and causes the patch to fail by delaminating and popping out of place.

Portland Cement Concrete Pavement (PCC) – A pavement constructed of Portland cement concrete with or without reinforcement. Conventional PCC pavements include JPCP, JRCP, and CRCP.

Potholes – Loss of surface material in an HMA pavement to the extent that a patch is needed to restore pavement rideability.

Preformed Compression Sealant – An extruded joint sealing material used for PCC pavement that is manufactured, ready for installation, and supplied in rolls. Preformed sealants incorporate an internal web design so that the material, when compressed and inserted into the sealant reservoir, remains in compression against the sides of the joint.

Present Serviceability Index (PSI) – A subjective rating of the pavement condition made by a group of individuals riding over the pavement. May also be determined based on condition survey information.

Present Worth – See Net Present Value.

Pumping – Ejection of fine-grained material and water from beneath the pavement through joints, cracks, or the pavement edge, caused by the deflection of the pavement under traffic loadings.

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RAP – Recycled Asphalt Pavement

Raveling – Wearing away of the pavement surface caused by the dislodging of aggregate particles and loss of asphalt binder. Also see Segregation.

Reactive Maintenance – Maintenance applied to restore a pavement to an acceptable level of service due to unforeseen conditions. Activities, such as pothole repairs, performed to correct random or isolated localized pavement distresses or failures, are considered reactive. Similar to Corrective Maintenance.

Recycling Agents – Organic materials with specific chemical and physical characteristics that are used in pavement recycling to address binder deficiencies and to restore aged asphalt material to desired specifications.

Reflection Cracking – Cracking that appears on the surface of a pavement above joints and cracks in the underlying pavement layer due to horizontal and vertical movement of these joints and cracks.

Regional Factor - Region specific. Deals with Drainage characteristics and terrain of area in question.

Reservoir – The part of a Portland cement concrete pavement joint that normally holds a sealant material, usually formed by a widening saw cut above the initial saw cut. Reservoirs may also be found in HMA pavements where joints are sawed and sealed above existing PCC pavements.

Retrofit Dowel Bars – Dowels that are installed into slots cut into the surface of an existing concrete pavement to restore load transfer.

Rideability – A measure of the ride quality of a pavement as perceived by its users or roughness measuring equipment.

Rigid Pavement - Pavement that will provide high bending resistance and distribute loads to the foundation over a comparatively large area.

Router – A mechanical device, with a rotary cutting system, that is used to widen, cut, and clean cracks in pavements prior to sealing.

Routine Maintenance – Maintenance work that is planned and performed on a routine basis in order to do the following:

- Maintain and preserve the condition of the highway system
- Respond to specific conditions and events that restore the highway system to an adequate level of service.

Examples include crack sealing, fog sealing, and repair of localized failed areas of pavement.

Rubberized Asphalt Sealant – A sealant, generally hot applied, that is composed of asphalt cement, various types of rubber or polymer modifiers, and other compounding ingredients used for pavement crack and joint sealing. Many grades and ranges of properties are available.

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Ultrathin Overlay – An HMA overlay over an existing HMA or PCC pavement, generally less than 25 mm (1 in) in thickness.

Unbonded Overlay – Increase in the pavement structure of an existing concrete or composite pavement by addition of jointed plain, or continuously reinforced concrete pavement placed on a separator layer (usually an asphalt layer) designed to prevent bonding to the existing pavement.

User Costs – Costs incurred by highway users traveling on the facility, and the excess costs incurred by those who cannot use the facility because of either agency or self-imposed detour requirements. User costs typically are comprised of vehicle operating costs (VOC), crash costs, and user delay costs. To be differentiated from agency costs.

Ultra-thin Whitetopping (UTW) – A thin (2 to 4 inch [50 to 100 mm]) PCC overlay over an existing HMA pavement. UTW is a functional overlay that provides a stable surface that is resistant to deformation from static, slow moving, and turning loads.

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Waterblasting – The use of a high-pressure water stream (8500 to 10,000 psi) to clean PCC. It may be used in PCC joint resealing to remove sawing laitance or in patching to produce a clean surface prior to placement of the sealer or patch material.

Water Table - The upper limit of the portion of the ground wholly saturated with water. (Webster) Typically, the first free water (static surface) encountered in an excavation. If there is an unsaturated soil zone known to exist at a lower elevation it may be referred to as the perched watertable.

Weathering - Changes in color, texture, strength, chemical composition or other properties of a natural or artificial material due to the action of the weather.

Wetland - Land which has the water table at, near, or above the land surface, or which is saturated for long enough periods to promote hydrophilic vegetation and various kinds of biological activity which are adapted to the wet environment.

Whitetopping - Concrete overlay pavement placed on an existing asphalt pavement.

Working Crack – A crack in a pavement that undergoes significant deflection and thermal opening and closing movements greater than 2 mm (1/16 in), typically oriented transverse to the pavement centerline.

Rutting – Longitudinal surface depressions in the wheel path of an HMA pavement, caused by plastic movement of the HMA mix, inadequate compaction, or abrasion from studded tires (such abrasion can also be observed on PCC pavements).

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Sandblasting – A procedure in which sand particles are blown with compressed air at a pavement surface to abrade and clean the surface. Sandblasting is a construction step in partial-depth patching and joint resealing.

Sand Grout - Grout mixture containing water, Portland Cement, and sand.

Sealant – A material that has adhesive and cohesive properties to seal joints, cracks, or other various openings against the entrance or passage of water or other debris in pavements (generally less than 76 mm (3 in) in width.

Sealant Reservoir – See Reservoir.

Sealing – The process of placing sealant material in prepared joints or cracks to minimize intrusion of water and incompressible materials. This term is also used to describe the application of pavement surface treatments.

Sealing Compound – See Joint Sealant.

Segregation – Separation of aggregate component of asphaltic or Portland Cement by particle size during placement.

Serviceability – Ability of a pavement to provide a safe and comfortable ride to its users. As such, it is primarily a measure of the functional capacity of the pavement.

Settlement – A depression at the pavement surface that is caused by the settling or erosion of one or more underlying layers.

Shoving – Localized displacement of an HMA pavement surface. Shoving is often caused by braking or accelerating vehicles.

Silicone Sealant – A type of joint or crack sealant compound either self leveling or non-sag in application characteristics, that is based on polymers of polysiloxane structures and cures through a chemical reaction when exposed to air.

Single Unit Trucks (SU) - Those are two-axle vehicles including buses. According to the FHWA Classification scheme. Single Units comprise vehicles from Class 1 through Class 5.

Slab Stabilization – Process of injecting grout or bituminous materials beneath PCC pavements in order to fill voids without raising the pavement.

Slippage cracking - Cracking associated with the horizontal displacement of a localized area of an HMA pavement surface.

Slurry – Mixture of a liquid and fine solid particles that together are denser than water.

Slurry Seal – A mixture of slow setting emulsified asphalt, well graded fine aggregate, mineral filler, and water. It is used to fill cracks and seal areas of old pavements, to restore a uniform surface texture, to seal the surface to prevent moisture and air intrusion into the pavement, and to improve skid resistance.

Soil cement - A construction material, a mix of pulverized natural soil with small amount of portland cement and water, and compacted to high density. Hard, semi-rigid durable material is formed by hydration of the cement particles.

Soil cement is frequently used as a construction material for road construction as a subbase layer reinforcing and protecting the subgrade. It has good compressive and shear strength, but low tensile strength and brittleness, so it is prone to forming cracks.

Soil Support - An index of subgrade strength. It is region specific, ranges from 2.0 to 4.5, Based on CBR (California Bearing Ratio) and converted to soil support value.

Soil Survey - A geotechnical exploration for roadways.

Spalling - Cracking, breaking, chipping, or fraying of slab edges.

Spalling, Compression – Cracking, breaking, chipping, or fraying of slab edges within 0.6 meters (2-ft) of a transverse crack.

Spalling, Sliver – Chipping of concrete edge along a joint sealant usually within 12 mm (0.5in) of the joint edge.

Spalling, Surface – Cracking, breaking, chipping, or fraying of slab surface, usually within a confined area less than 0.5 square meters (0.6 sy).

Stone Matrix Asphalt (SMA) – A mixture of asphalt binder, stabilizer material, mineral filler, and gap-graded aggregate. SMA's are used as a rut resistant wearing course.

Stress-Absorbing Membrane Interlayer (SAMI) – A thin layer that is placed between an underlying pavement and an HMA overlay for the purpose of dissipating movements and stresses at a joint or crack in the underlying pavement before they create stresses in the overlay. SAMI’s consist of a spray application of rubber- or polymer-modified asphalt as the stress-relieving material, followed by placing and seating aggregate chips.

Structural Condition – The condition of a pavement as it pertains to its ability to support repeated traffic loadings.

Structural Overlay – An increase in the pavement load carrying capacity by adding additional pavement layers.

Subbase - In highway engineering, subbase is the layer of aggregate material laid on the subgrade, on which the base course layer is located. It may be omitted when there will be only foot traffic on the pavement, but it is necessary for surfaces used by vehicles. Subbase is often the main load-bearing layer of the pavement. Its role is to spread the load evenly over the subgrade. The materials used may be either unbound granular, or cement-bound. The quality of subbase is very important for the useful life of the road. Unbound granular materials are usually crushed stone, crushed slag or concrete, or slate.

Subgrade - In highway engineering, a subgrade is the native materials underneath a constructed pavement. It is the foundation of the pavement structure, on which the subbase is laid. Subgrades are compacted, and are sometimes stabilized by the addition of cement or lime.

Surface Texture – The microscopic and macroscopic characteristics of the pavement surface that contribute to surface friction and noise.

Surface Treatment – Any application applied to an asphalt pavement surface to restore or protect the surface characteristics. Surface treatments include a spray application of asphalt (cement, cutback, or emulsion) and may or may not include the application of aggregate cover. Surface treatments are typically less than 25 mm (1 in) thick. They may also be referred to as surface seals, or seal coats or chip seals.

Swell - A hump in the pavement surface that may occur over a small area or as a longer, gradual wave; either type of swell can be accompanied by surface cracking.

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Terminal Serviceability – The lowest acceptable serviceability rating before resurfacing or reconstruction becomes necessary for the particular class of highway.

Thin Overlay – A HMA overlay with one lift of surface course generally with a thickness of 38 mm (1.5 in) or less.

Transverse Crack – A discontinuity in a pavement surface that runs generally perpendicular to the pavement centerline. In HMA pavements, transverse cracks often form as a result of thermal movements of the pavement or reflection from underlying layers. In PCC pavements, transverse cracks may be caused by fatigue, loss of support, or thermal movements.

Treatment Life – The period of time during which a treatment application remains effective. Treatment life is contrasted with Life Extension.

Two Component Sealant – A sealant supplied in two components which must be mixed at a specified ratio prior to application in order to cure to final properties.

Truck % - 24-hour trucks - Trucks cause the damage to the pavement structure and an accurate truck percentage will assist in reflecting the probable outcome.

References

Pavement Preservation Glossary of Terms, Foundation for Pavement Preservation, Austin, Texas, 2001

American Concrete Pavement Association Glossary, Skokie, Illinois, 2004 (online document)

Georgia Department of Transportation

A Appendix A: Lane Distribution for Multiple Lane Highways

(129 Counts in 6 States, 1982-83. Georgia participated) (Kher and Darter)

One Way ADT	2 Lanes (one Direction)		3+ Lanes (one-Direction)		
	Inner	Outer	Inner*	Center	Outer
2,000	6**	94	6	12	82
4,000	12	88	6	18	76
6,000	15	85	7	21	72
8,000	18	82	7	23	70
10,000	19	81	7	25	68
15,000	23	77	7	28	65
20,000	25	75	7	30	63
25,000	27	73	7	32	61
30,000	28	72	8	33	59
35,000	30	70	8	34	58
40,000	31	69	8	35	57
50,000	33	67	8	37	55
60,000	34	66	8	39	53
70,000	--	--	8	40	52
80,000	--	--	8	41	51
100,000	--	--	9	42	49

* Combined inner one or more lanes.

** Percent of all trucks in one direction (note that the proportion of trucks in one direction sums to 100 percent).

Georgia Department of Transportation

B Appendix B: The AASHO Road Test

At the time of its completion, the AASHO Road Test represented the most comprehensive development of the relationships between performance, structural thickness and traffic loadings of pavements. The results were limited by the scope of the test and the conditions under which they were conducted. Pavement design procedures that were based on the empirical results of the AASHO Road Test, were supplemented by existing design practice and available theory.

B.1 Roadbed Soil

A pavement structure is a layered system designed to distribute concentrated traffic loads to the subgrade. Preparation of the subgrade usually includes at least grading and compaction of the roadbed soils, and may include other means of providing support for the pavement structure.

Performance of the pavement structure is directly related to the physical properties and condition of the roadbed soil. The design procedures are based on the assumption that most soils can be adequately represented by means of the Soil Support Value (S) for flexible pavements or the modulus of subgrade reaction (k) for rigid pavements, and the design procedures compensate for poorer soils by increasing the thickness of the pavement structure.

Special provisions for unusually variable soil types and conditions may include: scarifying and re-compacting; treatment of an upper layer of roadbed soil with a suitable admixture; using appreciable depths of more suitable roadbed soils; over excavation of cut sections, and placing a uniform layer of select material in both cut and fill areas; or adjustment in the thickness of subbase at transitions from one soil type to another, particularly when the transition is from cut to fill sections.

Certain roadbed soils pose difficult problems in construction. These are primarily the cohesionless soils, which are readily displaced under equipment use to construct the pavements; and wet clay soils, which cannot be compacted at high water contents because of displacement under rolling equipment and require long periods of time to dry to a suitable water content. Measures that have been applied to alleviate such construction problems include: blending with other soils or adding admixtures to sands to provide cohesion, or in clays to hasten drying or increasing shear strength; and covering with a layer of more suitable select material to act as a working platform for construction of the pavement.

B.1.1 Pavement Drainage

Although the design procedure is based on the assumption that provisions will be made for surface and subsurface drainage, usually situations may require that special attention be given to design and construction of drainage systems. Drainage is particularly important where heavy flows of water are encountered (i.e., springs or seeps); or where soils are particularly susceptible to expansion or loss of strength with increased water content. Special subsurface drainage may include provision of additional layers of permeable material beneath the pavement for interception and collection of water, and pipe drains for the collection and transmission of water. Special surface drainage may require such facilities as dikes, paved ditches and catch basins.

B.1.2 Pavement Serviceability and Performance

The serviceability of a pavement is defined as the ability to serve high-speed, high-volume automobile and truck traffic. For the AASHO Road Test, a procedure was developed for periodic rating of the serviceability of the pavements. This procedure, known as the Present Serviceability Rating (PSR), consisted of the mean of individual ratings by a selected panel of men with long experience in all aspects of highway engineering, and as highway users. A scale with a range of 0 through 5 was established for present serviceability ratings, with a value of 5 for the highest index of serviceability and 0 as the lowest. A procedure was also developed for predicting the present serviceability rating from a combination of a series of physical measurements of the pavement. This combination of values was referred to as the Present Serviceability Index (PSI).

In order to develop the basic design equations from the AASHO Road Test data, it was necessary to establish the relationship between performance and pavement structural design, with performance being related to the ability to satisfactorily serve the traffic over a period of time. Performance of the AASHO Road Test pavements was described in terms of the serviceability index at the time of completion of construction and at some later time subsequent to construction. This serviceability-performance concept is the basic philosophy of this design guide and pavements may be designed for the level of serviceability desired at the end of the selected traffic analysis period or after exposure to a specific total traffic volume. Selection of the terminal serviceability index (p_t) is based on the lowest index that will be tolerated before resurfacing or reconstruction becomes necessary. An index of 2.5 was suggested as a guide for major highways, and 2.0 for highways with lesser traffic volumes.

B.1.3 Traffic Loading

The basic equations developed from the results of the AASHO Road Test were based on traffic that consisted of multiple applications of identical vehicle loads on each of the test loops. In order to be applicable to the design of pavements, these equations must be extended to use with mixed traffic; i.e., the random mixture of vehicles with different axle loads and number of axles that constitute normal highway traffic. The procedure used in this design Guide is to convert the varying axle loads to a common denominator, and to express traffic as the sum of the converted axle loads. The common denominator used is an18-kip (80kN) single-axle load. Thus, traffic is expressed as equivalent 18-kip (80kN) single-axle loads.

The prediction of traffic for design purposes must rely on information from past traffic, modified by factors for growth. Most states accumulate past traffic information in the form of loadometer data in the format of the FHWA W4 loadometer tables, which are tabulations of numbers of axles observed within a series of load groups, with each load group usually a 2,000-lb (8.9kN) interval. These tabulations are in a convenient form for conversion, since the number of axles in each load group may be multiplied by an appropriate factor for conversion to equivalent 18-kip (80kN) single axle load applications for the load group, and a summation of these for all load groups is the equivalent 18-kip (80kN) single axle load application that represents the total traffic for the design period. Note that the equations developed in this Guide were based on the application of a maximum number of loads during a two year period at the AASHO Road Test.

Predictions of traffic are made for some convenient period of time, known as the traffic analysis period. The traffic analysis period often used is 20 years, which is also a common period used in traffic predictions often used for geometric design. However, any period may be used with this design guide because traffic is expressed as daily or equivalent 18-kip (80kN) single axle load applications. Regardless of the traffic analysis period used, the total equivalent 18-kip (80kN) single axle load application is the total traffic that the pavement can be expected to carry from the time of construction to the time when the serviceability is reduced to the selected value. Thus, if traffic is underestimated, this time may be less than the traffic analysis period, and, conversely, if the traffic is over-estimated this time can be expected to be longer. Neither the traffic analysis period nor the time a pavement reaches its terminal serviceability index (P_t) should be confused with pavement life. Pavement life may be extended by periodic renewal of the surface. Also surface renewal may be necessary for reasons other than restoration of serviceability (such as renewal of antiskid properties or rejuvenation of weathered surfaces).

The equivalent axle loads derived from many prediction procedures represent the totals for all lanes for both directions of travel. This traffic must be distributed by direction and by lanes for the purpose of design. Directional distribution is usually made by assigning 50 percent of the traffic to each direction, unless special conditions, i.e., traffic diagrams, warrant some other distribution. In regards to lane distribution, 100 percent of the traffic in each direction is usually assigned to all lanes in that direction for the purpose of structural design.

B.1.4 Pavement Environment

The significant environmental factors affecting long term pavement performance are annual rainfall (moisture) and temperature.

Pavement and subgrade moisture conditions exert a major influence on the performance of roads. In pavement design it is important to be able to recognize ways by which moisture may enter the pavement or subgrade and to determine measures needed to control moisture movement. Moisture changes usually result from one or more of the following effects:

- Seepage from higher ground
- Fluctuations in water table level
- Infiltration of water through the surface of the pavement as well as shoulders
- Transfer of moisture in liquid or vapor states

The temperature environment has a major influence on the performance of pavements. Asphalt becomes stiff and brittle at low temperatures while it is soft and visco-elastic at higher temperatures. Permanent deformation in asphalt at higher temperatures may occur, although this is generally considered as a mix design problem and not a pavement design problem.

Concrete expands and contract due to temperature changes. This fluctuation in temperature causes shrinkage or expansion of the slab, and induces thermal and other internal stresses in the slab. The curling at the slab corners is a result of those internal stresses.

B.1.5 Concrete Material Properties

The average flexural strength for concrete on the AASHO Road Test was about 690 psi (4.8MPa) at 28 days. In order to make the design procedure applicable to concrete of other flexural strengths, it was necessary to develop a scale for this purpose.

The modulus of rupture (Sc) at 28-days as determined by the test procedure specified in AASHTO Designation T-97, using third-point loading, is the basis for determining concrete flexural strengths.

A static modulus of elasticity (Ec) of 4,200,000 psi (29GPa) was the average value for concrete on the Road Test and was used in developing the design charts.

C Appendix C: Pavement Condition Evaluation Guidelines

C.1 Purpose

The purpose of this Appendix to the GDOT Pavement Design Manual is to provide a summation of pavement evaluation and design requirements for use by professionals who are engaged in the preparation of pavement evaluations and pavement designs for the Georgia Department of Transportation (GDOT).

C.2 Preface

A project's scope should be clearly understood before undertaking a pavement evaluation. The project scope is a description of the parameters of the project and can be found in the project Concept Document or Plans. The Concept Document or Plans define the problem the project is intended to address, a proposed solution, project limits, and funding information. The Concept Document is developed at the time of the project's initial conception. The Concept Document or Plans for Pavement Rehabilitation type projects are based on an assessment of the condition of the existing pavement and the construction history for the project. During project development the scope can change as new information is obtained. It is important for the GDOT Designer to keep in contact with the Project Manager; or in the case of Consultant Designers, to keep in contact with the Consultant Project Manager.

The pavement evaluation, analysis, and design discussed herein must be developed by, or under the direct supervision of, a Professional Engineer (civil discipline) registered in the State of Georgia. The engineer will place their professional seal on the pavement evaluation/design report and will be the Engineer of Record for that design.

Throughout this guide, there are references to responsibilities of the "Designer". Designer means the GDOT technical staff responsible for pavement designs for "in-house" projects completed by GDOT. For out-sourced projects, "Designer" means the professional consultant under contract with GDOT to provide both Pavement Evaluation and Pavement Design services.

Pavement evaluation/design recommendations and all supporting documentation, including design assumptions, background information, and field data, must be compiled and submitted for review in a bound Evaluation & Design Report. The report shall be a logical presentation of all the materials that have been gathered leading to the design recommendations that are being presented as a summation of all the work.

All pavement designs proposed for GDOT must use the most cost-effective design that meets the objectives of the project and complies with all applicable design standards. All pavement designs for GDOT, prepared by Consultants, and submitted for review, must be developed using the AASHTO Interim Guide for Design of Pavement Structures 1972, Chapter III Revised, 1981.

The GDOT State Pavement Engineer, or other qualified staff member, shall review pavement evaluations and designs for structural adequacy and compliance with the guidelines set forth in this document along with other applicable GDOT documents.

The user should keep in mind that this document will be updated periodically as required. The document will be expanded to provide additional state of the art information. Questions regarding any of the information presented in this guide may be directed to:

- AJ Jubran, P.E. – State Pavement Engineer @ 404-363-7582 Abdallah.Jubran@dot.state.ga.us
- Moussa Issa - Pavement Design Engineer @ 404-363-7620 Moussa.Issa@dot.state.ga.us

C.3 Preliminary Pavement Evaluations

C.3.1 When What

A "Preliminary Pavement Evaluation Report" is required during Concept Development/Validation, Phase I of the project. This evaluation is limited to data acquisition and preliminary analysis to develop preliminary pavement design recommendations. A preliminary evaluation does not include actual field and/or laboratory testing as typically required for a complete pavement evaluation. The requirements for a complete pavement evaluation are discussed in detail in subsequent sections of this appendix.

See Chapter 9.1 for reference.

C.3.2 How

The primary steps involved in a preliminary pavement evaluation include the following:

Gather readily available data on the original pavement design and any maintenance of the pavement since construction. A discussion of such data, including likely sources, is provided in subsequent sections of this Appendix.

Conduct a "Windshield Survey" of the project alignment to develop a good understanding of the existing pavement, shoulders, ditches, etc. This step will include a very generalized assessment of the pavement type (asphalt or concrete), condition (good, fair, poor), drainage and evidence of past maintenance.

Obtain a preliminary SSV from the table "Average Typical Soil Support Values for Estimating Purposes Only" included in Appendix G of this manual.

Perform a preliminary pavement design utilizing project design traffic information and loading provided.

The report should include a written summary and be submitted in PACES or CPACES format. (Draft format included). It should be attached to the Concept or Concept Validation Report. The written summary report should include the following:

- Project Identification (GDOT Project Number & Location)
- Historical Overview of Project (if readily available)
- Summary of visible findings from the "Windshield Survey"
- A listing of the parameters used for the preliminary pavement design
- The actual preliminary pavement design

C.4 Final Pavement Evaluations

C.4.1 General

A "Final Pavement Evaluation Report" is required during Preliminary Plan Preparation/Development, Phase IV of the project. The evaluation is an extensive study which incorporates data regarding original design, on-going maintenance and future traffic loading for the project. The following sections detail the necessary steps required to complete a final pavement evaluation.

See Chapter 9.2 for reference.

C.4.2 Historical and Design Data Collection Guidelines

The first step in a pavement evaluation would be the collection of all available data on the history and future intended use of the project. This section provides guidance on data collection and covers both office and field data. The intent of this section is to provide resource information such as what is available and how to obtain the information, including construction history, pavement condition, and traffic data. This section of the Appendix also provides guidance on the minimum acceptable levels of fieldwork required for the development of pavement evaluations and designs.

Construction and Maintenance History

Construction and maintenance history are essential in developing a field investigation strategy, determining the existing material types and depths, and evaluating the performance of existing materials. GDOT usually maintains a record of as-constructed drawings stored in The Design Store, accessible only at GDOT office. Useful information from these drawings includes the cover sheet, details, typical pavement sections and summary. These plans are valuable resources, but the Designer is cautioned that the information contained in the files is not always complete. Also, maintenance work is usually not included in these plans. Maintenance work records should be available from the Area Maintenance Engineer's Office or the District Maintenance Engineer's Office.

Another source of data is the GDOT Pavement Condition Evaluation System (PACES) database and C-PACES database. PACES and C-PACES can provide construction history and pavement condition information for flexible and rigid pavements. Summary information for each section of highway can be obtained from the Pavement Condition Report. The report, published every year, provides condition information on each section of highway. Consultants may order hard copies of the PACES and C-PACES manuals on how to conduct the condition survey and the rating procedures from The Office of Maintenance.

Field Reconnaissance

A field reconnaissance is a site visit for the purpose of determining the type and extent of field investigation work required on the project and specific locations the Designer (Pavement Evaluation Engineer) requires detailed evaluation of the pavement, either through destructive or non-destructive means. In addition to planning the field investigation work, it gives the Designer an opportunity to determine the requirements for traffic control that would be required during the field evaluation. A walk through the project limits, by the engineer, would identify locations of visible distress warranting additional or more in-depth testing and investigation.

If the pavement evaluation is to be completed by a Consultant, the Field Reconnaissance may have to be completed before contract execution/Notice to Proceed is given for large or complex projects where the pavement evaluation is critical to the project. This could be required in order to develop a realistic project scope and estimated cost/budget to complete the actual pavement evaluation. If a Field Reconnaissance is not performed prior to development of the pavement evaluation scope of services, then a Special Studies budget allocation is recommended to cover unforeseen tests (either field or laboratory) that could not be predicted without the Field Reconnaissance by the Consultant. Special Studies budgets should not be treated as part of the authorized initial scope of services for a Consultant. Instead, the Special Studies budget should be considered a second phase, which requires approval from GDOT's Pavement Management Branch and the GDOT Project manager before proceeding.

Traffic Data

Traffic data is an important component of any pavement evaluation and design analysis. These data typically consist of 24-hour traffic counts, truck percentages, along with percentages of single units and multiple units. Traffic information can be obtained from the Office of Environment and Location or through the GDOT Project Manager. It is essential that the growth rate and traffic data for ESAL calculations on all projects requiring a pavement design be obtained from GDOT.

C.4.3 Field Testing / Investigation Guidelines

General

The intent of this section is to provide guidance on the type and extent of field investigation required for the development of pavement design recommendations. The guidance provided herein should be considered as a starting point and is intended to represent the minimum level of field investigation required. As each project will be unique, the field investigation plan must be adjusted to provide adequate information for addressing the needs of the project

The following sub-sections outline the field investigation requirements for GDOT projects. Each sub-section discusses the requirements for a particular type of testing, such as deflection testing, coring, distress surveys in accordance with PACES and C-PACES Guidelines, and additional testing that the Consultant may deem appropriate.

GDOT defines new work as the construction of new pavement, including widening of existing facilities and new alignments.

Pavement rehabilitation is defined as any work on an existing facility that does not change the geometry or add capacity, and includes work such as inlays, overlays, or reconstruction.

A review of the Concept Document or Plans and a field reconnaissance are the first steps in developing the field investigation plan. The field reconnaissance provides the Designer with the opportunity to evaluate the project for the types of investigative procedures that may be required along with the testing and sampling locations and frequencies.

Traffic Control Associated with Fieldwork

Traffic control must be conducted in accordance with the latest version of “Traffic Control on State Highways for Short Term Work Zones", published by the Georgia Department of Transportation’s Office of Maintenance. In the case of Contractor field investigations, traffic control must be conducted in accordance with the contract documents and other GDOT applicable documents.

Non-Destructive Field Testing

Non-destructive field-testing for pavement evaluation purposes is most often used when large volumes of data are needed, when safety concerns limit access to the pavement under evaluation and/or when structural information and performance data of an entire pavement section is needed. There are many forms of non-destructive testing that could be addressed in this section; however, the discussion included herein is limited to those tests most routinely utilized by GDOT. Please note however that the Designer, on a project-by-project basis, is encouraged to develop a scope of services relative to the planned fieldwork that provides the best data for pavement evaluation and design.

PACES and CPACES Surveys

- **Pavement Distress Surveys (PACES and CPACES)** are an integral part of a successful pavement rehabilitation project. Pavement distresses are defects in the pavement surface such as ruts and cracks. Proper distress identification helps the designer determine the mode of failure such as, whether the distress is due to load related factors or environmental effects. In addition the distress surveys help the engineer develop the field investigation plan, determine if reflective cracking would be a factor in the rehabilitation performance, and are a primary factor in locating areas that require localized repairs. When combined with other data collected on a project such as cores and deflections, distress surveys are very important in assessing the pavement rehabilitation needs.

- GDOT has adopted pavement distress definitions based on the Pavement Condition Evaluation System (PACES) for flexible pavement and Concrete Pavement Condition Evaluation System (CPACES) for rigid pavements, for both network and project level pavement distress surveys. The definitions and measurement protocols have been modified to better suit conditions founds in Georgia. The Office of Maintenance of the GDOT maintains manuals for both asphalt and Portland cement concrete pavement. See Appendix E for the PACES manual and Appendix F for the CPACES manual.
- The minimum information required in a distress survey includes:
    - Types of distress
    - Severity of distress
    - Extent of distress
    - Location of distress
- For asphalt concrete and PCCP pavements, a simple form may be used. For reinforced and jointed plain concrete pavements, it is strongly recommended that the designer create a crack map for conducting the distress survey. The crack map allows the designer to identify and locate distresses in individual slabs. This information can be used later in determining repair and under-sealing quantities, as well as for marking the repair areas in the field.
- Rut depths must be measured on all flexible pavement projects at a maximum of ¼ -mile increments. Ruts must be measured in all wheel paths using as reference the stringline used in Bituminous Construction,a 5 or 6 ft straight edge, or Rut Depth Measurement Device. Measurements must be estimated to the nearest 1/8 in (3 mm). The average rut depth and standard deviation for each wheel track must be reported. A summary of the rut measurements must be provided in the design report in accordance with the Deliverables section of this guide.

Photographic Documentation

Photographs are used to provide a visual record of conditions at the time the survey is conducted. Photos are suggested for new work sections and are left to the Designer’s discretion, but are required on all rehabilitation projects. When photographs are taken on a given project: A maximum spacing of ½ mile is suggested.

Photographs must be taken using 35 mm film or with a digital camera (if 35 mm film is used, digital processing is preferred). Photos must be taken looking in both directions of travel of the lane at each location.

Copies of all photos must be submitted in accordance with the guidelines provided in the Deliverables section of this guide. Photos must be arranged by mile point and labeled with the date, mile point and direction of the photograph.

The following protocol should be followed in labeling photographs and in submitting digital photographs using a jpeg or a gif format.

Photograph Type	Label Protocol
County Number	A three digit number
Route Number	A four digit ID followed by a two character suffix
Route Code	State or County or other code for the route
Direction of Travel	E or W, N or S
Lane of Travel	A one-digit number
With Traffic or Facing Traffic	W or F

Thus SR 101 in County 053 traveling south in lane 2 facing traffic, with spaces added to emphasize the various components of the numbering and labeling protocol, would be: **053 0101 00 1 S 2 F.jpg**

Photographs should be included in the Pavement Evaluation report on a 3½" floppy disk or CD-ROM.

Pavement Drainage Survey

The presence of moisture is a primary cause of distress or failure of all pavements. Therefore, a drainage survey is an important component of pavement evaluation. Moisture conditions are caused externally by the climatic conditions and internally by the properties of the materials composing the pavement structure. The severity of damage caused by excessive moisture will influence the decision on which rehabilitation strategy to select. Since moisture problems can exist in any layer of the pavement structure, more than visual observations may be needed. Cores and nondestructive testing may need to be conducted. It is necessary to determine which material is responsible for the moisture-related damage and if an economical rehabilitation to correct the problem is to be initiated. Not identifying and correcting the problem could lead to a failed project. Valuable tools in this evaluation are the as-built plans and maintenance documents. In addition to determining if the pavement structure is freely draining and moisture-resistant, the entire roadway section should be evaluated including:

- Are the ditchlines free of standing water? If not, how high does it stand and will it infiltrate the pavement structure?
- Are ditchlines and pavement edges clear of the type of growth that would indicate excessive moisture?
- After a rain, is water standing in the joints or cracks?
- Is there standing water adjacent to the pavement or on the shoulder?
- If there are drainage outlets including under-drains, are the outlets clear, at the proper elevation, and working?
- Are drainage inlets clear and cross slopes adequate to remove the water from the pavement surface?
- Are joint and crack sealants in good condition and preventing surface water infiltration?
- Are there signs of pumping, such as pavement discoloration or the presence of fine material at joints or pavement edges?
- Recommending drainage improvements to the pavement structure can be a very expensive item and should be carefully evaluated and documented.

Falling Weight Deflectometer (FWD) Testing

General

The Falling Weight Deflectometer (FWD) is used to measure deflections of the entire pavement section in accordance with ASTM-D4694 by applying loads to the pavement and measuring the deflections in at least 7 locations along the test section.

Deflections

Pavement deflection testing shall be conducted on existing pavements identified for rehabilitation or widening. This data may be used to:

- Determine statistically different performing pavement sections and sub sections in order to refine destructive sampling plans
- Determine the soil and subgrade strength
- Assess the structural capacity of pavement structures
- Estimate the structural capacity of the pavement structure and the material properties of the individual pavement layers
- Estimate the load transfer ability of joints in jointed rigid and composite pavements

Guidance available in the AASHTO Guide for Design of Pavement Structures, 1993 and Mechanistic Empirical Pavement Design Guide (NCHRP 1-37A) incorporates deflection testing and analysis as an important part of the evaluation of existing pavement structures. Various methods are used to evaluate deflection data ranging from statistical assessment through determination of pavement layer properties.

Deflection testing performed for Georgia roadway projects is used to assess the following:

- Asphalt concrete pavements.
- Variation in pavement response based on cumulative sum of maximum deflection and cumulative sum of deflection at the 60 inch offset
- If pavement thickness data are available the following should also be calculated:
  - Effective Structural Number and subgrade resilient modulus
  - Elastic modulus of each of the structural layers (at non-distressed locations).

Concrete pavements

- Variation in pavement response based on cumulative sum of maximum deflection and cumulative sum of deflection at the 60 inch offset
- Load transfer across joints (across transverse joints in wheel path).
- Void intercept (variable load corner deflection method).
- Impulse Stiffness Modulus, ISM
- If pavement thickness data are available the following should also be calculated from center slab deflections:
  - Concrete elastic modulus
  - Modulus of subgrade reaction.
  - Specific calculation methods are presented in the reporting section below.

- Deflections must be measured with an impulse device such as a Falling Weight Deflectometer (FWD), meeting the requirements of ASTM-D4694-96 (2003). The device must have a minimum of 7 deflection sensors and these must be located as summarized in Table C-1. Load levels for testing will vary according to pavement type and are summarized in Table C-2. Deviations from the applied loads and sensor spacing must be approved in writing by the GDOT Pavement Engineer prior to field activity.

Pavement Type	Sensor Spacing
Flexible	0”, 8”, 12”, 18”, 24”, 36”, 60”
Rigid and Composite	-12”, 0”, 12”, 24”, 36”, 48”, 60”

TABLE C-1 - DEFLECTION SENSOR RADIAL OFFSET

Load Package ID	Pavement Type	HMA Thickness	Test Type	Load Package and Sequence
1	Flexible	< 4”	Basin	BB2B2
2	Flexible	> 4” and < 8”	Basin	BB2C3
3	Flexible	> 8”	Basin	BB2D4
4	Rigid	N/A	Basin	BC3D4
5	Rigid	N/A	Joint	BB2C3D4
6	Composite	N/A	Basin	BC3D4
7	Composite	N/A	Joint	BB2C3D4
8	Subgrade, Base	N/A	Basin	AA1B2

TABLE C-2 :TARGET LOAD LEVELS FOR DEFLECTION TESTING

Note: Letters are load heights, and data are not recorded. Numbers are load heights and data are recorded.

- A = height 1, target 6,000 lbf; B = height 2, target 9,000 lbf, C = height 3, target 12,000 lbf; D = height 4, target 15,000 lbf. Actual loads shall be within 3% of target.
- The FWD comprises several different measuring elements that must each be calibrated. Reference calibration for deflection sensors and load cell must be performed annually. Relative calibration of deflection sensors must be performed immediately prior to a project or monthly whichever is the greater frequency. Temperature measuring devices must be verified annually. Prior to beginning work on a project, and as needed or directed, the FWD's Distance Measurement Instrument must be calibrated to insure proper distance measurement.
- Written documentation by the reference calibration center is required to be submitted to the Pavement Engineer (GDOT) to show that the calibration has been conducted successfully within the 12-month period prior to its use on a project. Copies of supporting documentation for relative calibrations of deflection sensors and distance measuring equipment shall be made available to the Pavement Engineer as well. Test types are summarized in Table 3. Depending on the type of pavement structure, the location, load plate size, and drop sequence all vary. Basin testing uses the sensor spacing in Table 1, load levels based on pavement type from Table 2 and plate type and test location as shown in Table 3.

Pavement Type	Test Type	Load Plate Type	Test Point Location in Lane
Flexible	Basin	Small (6" radius)	Right Wheel Path
Rigid Jointed	Basin	Small (6" radius)	Mid Slab
Rigid Jointed	Joint	Small (6" radius)	Right Wheel Path/At Joint
Rigid - Jointed (Slab Corner)	Corner	Small (6" radius)	Corner of Slab/ At Joint
Rigid – CRCP	Basin	Small (6" radius)	Right Wheel Path
Composite	Basin	Small (6" radius)	Mid Slab (if joint reflection cracks are present)
Composite	Joint	Small (6" radius)	Right Wheel Path/At Joint Reflection Crack (if present)
Subgrade/Unbound Base	Basin	Large (9" radius)	Planned mid lane

TABLE C-3 SUMMARY OF TEST TYPES BY PAVEMENT TYPE

Asphalt Concrete Pavement

Mainline Pavement

Testing will be conducted in the right lane, right wheel path. If there is extensive wheel path cracking then offsetting to the mid-lane path would be acceptable but should be noted as an exception for purposes of reporting the results. The number of tests needed to characterize the project shall govern test spacing. As a minimum, 20 locations per lane mile should be obtained in order to collect sufficient data for statistical soundness. Consideration shall be given to reducing this spacing in urban areas or areas of localized structural failure. In multi-lane sections, deflections must be taken in both directions in accordance with the above requirements. The Designer shall use professional judgment to consider additional testing in multi-lane sections if the pavement condition and/or construction history varies significantly.

Shoulders

If project plans call for the possibility of traffic being routed onto the shoulder an assessment of the structural condition is needed. Deflections must be measured on the shoulder at a maximum spacing of 250 ft (76 m) to help determine if the shoulders are structurally sufficient to carry travel lane traffic during construction. If the project is intended to include existing shoulder pavement into the mainline pavement then the data can be used to assess structural strengthening necessary to meet design requirements. If the existing pavement is to be structurally overlaid in addition to widening, deflection testing is required. If widening is only to increase shoulder width and will not normally carry traffic, deflection testing is not required.

Rigid Pavement

Deflection testing requirements for PCC pavements are different from asphalt concrete pavements and are dependant on the type of PCC pavement; continuously reinforced or jointed. Deflection measurements on PCC pavements are used to determine overall stiffness, material properties, load transfer at the joints, and for void detection.

Continuously Reinforced Concrete Pavement

For the determination of material properties, testing should be conducted in the mid lane between the wheel paths. A testing frequency adequate to provide a statistical representation of the response properties along the project is required. Care should be taken to avoid placing the load plate on transverse cracks, if possible.

Testing at transverse cracks to determine load transfer should only be considered at cracks that are spalled or are faulted. Transverse cracks are a natural occurrence in CRCP pavements and may be spaced as close as 3.5 ft from each other. Badly spalled, highly faulted transverse cracks and punchouts should not be tested.

Jointed Plain and Reinforced Concrete Pavement

For jointed concrete pavements (JCP - reinforced and plain concrete pavement), deflection measurements are required to determine material properties, load transfer at the joints, and for void detection.

For the determination of material properties, testing should be conducted in the mid slab. A testing frequency adequate to provide a statistical representation of the material properties along the project is required but no less than 20 per directional mile. The sensor spacing presented in Table 1 for Rigid pavement should be used.

As indicated in Table 1 the sensor spacing for Rigid pavement testing (in particular for load transfer and void detection analysis) is different than the Basin configuration to allow testing on either side of a transverse joint. For this testing, a sensor must be placed at a distance of -12 inches (300 mm) behind the load plate. There are two ways to accomplish this. The first is to move the sensor located at 18 inches from the load cell to the new location. The alternative method is to add a sensor at the -12 inch offset.

Each slab tested will have a Basin test performed in the center of the slab. A minimum of 50% of slabs having Basin tests will also have load transfer testing performed. For load transfer testing the Basin test is first performed. Then the FWD is positioned so the load plate is placed on the same slab but near the joint (within 1 inch) in the right wheel path so that the sensor located at 12 in (300 mm) is on the unloaded slab. After this test the FWD is moved forward such that the load plate is on the leave slab and the -12 inch sensor is on the slab where the Basin test was performed.

Void detection testing shall be conducted on jointed pavements having noticeable faulting and evidence of pumping or as directed. Void detection testing shall be the variable corner deflection approach. The FWD is positioned so the load plate is placed on the slab corner near the transverse joint (within 1 inch) and within 12 inches of the slab edge so that the sensor located at 12 in (300 mm) is on the unloaded slab. After this test the FWD is moved forward such that the load plate is on the leave slab and the -12 inch sensor is on the previous slab. A testing frequency adequate to provide a representative sample of the void detection results on the section is required.

Due to the effects of temperature on the behavior of concrete slabs, all testing must be done when the PCC surface temperature is between 50 and 80°F (10 to 27°C), generally prior to 11 a.m. or if cloudy conditions exist such that surface temperatures are not excessive due to sunlight. A testing frequency adequate to provide a representative sample of the load transfer on the section and the percentage of slabs with voids is required.

Composite Pavement

For composite pavements, AC over PCC, the location of transverse joints may be inferred from the presence of transverse (reflection) cracks. If transverse cracks are present, Basin and Load Transfer testing should be performed. If reflection cracks are not prominent Basin testing should be performed at a spacing of no less than 20 per directional mile.

Selection of Test Locations

When selecting locations to test in the field, consideration shall be given to the condition of the pavement. Cracks in pavements affect deflections considerably. Every effort shall be made to take deflections at least 6 ft from cracks or at least to note the presence of cracks within the radius of the sensors (within 60 inches of the load plate). Methods used to determine material properties from deflection data rely on “ideal” conditions. As such, if discontinuities are present within the deflected pavement the data violates the assumptions for the model used to assess the properties.

Consideration shall also be given on JCP pavements when selecting joint test locations. If joints that are severely spalled, faulted or contain corner cracks or breaks are to be repaired they should not be tested.

Reporting

Deflection testing and analysis of the data should be documented in an evaluation package containing the following elements.

- Transmittal Letter
- Project Description
- Data Collection Scope
- Analysis Results
- Discussion
- Appendix – electronic files of FWD data

Deflection Analysis Reports

Asphalt Concrete Pavement:

- Plot of normalized maximum and normalized minimum (60 inch offset) deflection vs. location
- Plot of cumulative sum of maximum deflection vs. location
- Tabular summaries of: (only if thickness information is available)
- Pavement layer thicknesses (asphalt concrete, base)
- AASHTO SN effective and uncorrected subgrade resilient modulus
- Asphalt modulus, base course modulus, subgrade modulus, from back calculation. OMR needs to be contacted to determine which back calculation program is being used at the time.
- Asphalt modulus corrected to 68˚F, mid depth temperature of asphalt concrete (BELLS3 model)

- Deflection data: distance (ft), load (lbf), deflections, surface temperature, time, comments

Rigid Pavement

- Plots of normalized maximum deflection of Basin tests vs. location
- Plot of cumulative sum of maximum deflection of Basin tests vs. location
- Plot of LTE vs. location
- Plot of void intercept vs. location
- Tabular summaries of: (only if thickness information is available)
- Pavement layer thicknesses (PCC, base)
- AASHTO PCC elastic modulus, Sc' (AREA method)
- AASHTO modulus of subgrade reaction (AREA method)
- Deflection data: distance (ft), load (lbf), deflections, surface temperature, time, comments, slab bending factor B, LTE, Void intercept

Composite Pavement

- Plot of normalized maximum deflection of Basin tests vs. location
- Plot of cumulative sum of maximum deflection of Basin tests vs. location
- Plot of LTE vs. location
- Tabular summaries of: (only if thickness information is available)
- Pavement layer thicknesses (AC, PCC, base)
- AASHTO PCC elastic modulus, Sc' (AREA method)
- AASHTO modulus of subgrade reaction (AREA method)
- Asphalt modulus (estimated from temperature using Witczak equation)
- Deflection data: distance (ft), load (lbf), deflections, surface temperature, time, comments, bending/compression factor B, LTE

Ground Penetrating Radar (GPR) Testing

GPR is a geophysical non-intrusive technique, operates by transmitting short pulses of electromagnetic energy into the pavement using an antenna attached to a survey vehicle traveling at normal driving speed. It measures changes in dielectric properties of pavement layers and the velocity of wave propagation within those layers. It is effective at detecting stripping in HMA layers where the deterioration is at a moderate to advanced stage. HMA that has experienced stripping has higher moisture contents and/or higher void ratios.

The result of GPR data analysis is that stripping can be identified as a layer within the asphalt structure. This analysis can be quantified and plotted on a linear or plan area plot. The key to establishing this algorithm is to identify the appropriate thresholds for each of the observed asphalt layers. Establishing the algorithm is site dependent. It also requires substantial experience interpreting GPR data.

The most common applications for GPR have been for thickness measurements associated with rehabilitation design and for pavement structure inventory data input to pavement management systems. These data include pavement condition diagnosis such as detection of voids or stripping in HMA layers. The GPR technique has been found to be effective in full depth HMA, HMA overlay over old flexible pavement, and HMA overlay over rigid pavement. The GPR method will cover pavement sections in a fraction of the time required for taking cores and provides thousands of data points compared to the limited number of data points provided by coring.

GPR depth of penetration is limited to about one meter when using the most powerful equipment presently available, a 1 GHz horn antenna. Recent studies have shown that base layer thickness for cement treated bases cannot be measured with reliability.

Seismographic Techniques

Moisture damage and/or actual stripping typically will result in significant decrease in the modulus values of Hot Mix Asphalt (HMA) mixtures. Young's modulus, E, is the primary parameter of interest to pavement engineers and it can be determined from shear modulus through Poisson's ratio.

Utilizing a Portable Seismic Properties Analyzer (PSPA), a shear modulus profile is obtained by measuring the dispersion of shear waves. The PSPA is also utilized for assessing the impact of stripping on the variation of HMA modulus with depth. Two methodologies are available to evaluate bound layer properties:

- **Ultrasonic Surface Waves (USW) –** measures the stiffness of the bound layer. It is used to obtain the modulus of the HMA.
- **Impact Echo –** measures the thickness of the bound layer or identifies delaminated layers

When HMA is impacted with a point source, body and surface waves propagate in the material. Because surface waves propagate along a cylindrical front, the depth of inspection can be controlled.

The Impact Echo method primarily provides information about the thickness of a layer. However, it is not applicable to relatively thin layers and layers where the difference in moduli of adjacent layers is small.

With the PSPA, the average modulus of the exposed surface layers can be estimated in the field within a few seconds. The variation in modulus with depth can be qualitatively evaluated with further computer data analysis. The response of a viscoelastic material, HMA, is dependent on the loading frequency and temperature. The general practice has been to perform the testing at various temperatures with similar loading frequencies.

Inertial Profiler

The function of an inertial road profiler is to collect accurate longitudinal profiles to generate statistics that summarize the character of the profiles, particularly those that contribute to road roughness. The most widely used statistic to summarize the character of a profile is the International Roughness Index (IRI). Inertial profilers, normally mounted in a vehicle, can perform precise profile measurements at speeds up to 65 mph. The system generates a profile type plot with “defect locations” and “must grind” lines that indicates where the roughness exists and what corrective action to take.

Road profiles change over time and from season to season. Understanding seasonal influences is important for determining the optimum times to conduct yearly distress surveys.

The equipment should meet the requirements of ASTM E950 Class I profiling device.

Inertial Profilers yield satisfactory, reliable results on asphalt and concrete surfaces. This work would be performed in accordance with GDT126 “Test Method for Determining The Ride Quality and Smoothness of a Pavement Surface Using a Road Profiler.” Individual road reports become part of the annual Highway Performance Monitoring System (HPMS) Report.

Dipstick/Walking Profiler

It is a lightweight manually operated integrated data collection instrument that is very sensitive to high frequency roughness. The operator “walks” the Dipstick along a survey line by alternately pivoting the instrument about each leg, automatically recording each elevation difference reading. The Walking Profiler collects data at true walking speed gathering surface profile data and generating ride quality statistics and precise depictions of localized roughness. These instruments are excellent methods for measuring transverse profiles of roads.

The Dipstick is included in the LTPP Program Directive P-10 in comparison to the Inertial Profiler.

C.4.3.1 Destructive Field Testing

Destructive field-testing for pavement evaluation purposes is a basic component of a pavement evaluation and is most often used. Please note however that the Designer, on a project-by-project basis, is encouraged to develop a scope of services relative to the planned fieldwork that provides the best data for pavement evaluation and design.

Pavement Cores

Pavement depths are usually determined by either cutting an asphalt concrete (AC) core or from an exploration hole. Cores must be of sufficient size to determine the condition of the pavement layers and crack depths. In addition, the Designer must consider the requirements of any laboratory testing that may be conducted on cores. GDOT typically collects 6 in (100 mm) diameter core samples. If pavement cracking is a concern, the Designer must arrange for some of the cores to be cut through the cracks to evaluate the extent and severity of the cracking.

For the widening of existing facilities, cores must be taken on the shoulders to determine the depth, type and condition of existing materials. This requirement is for minor shoulder widening and where the existing shoulder will be incorporated into a travel lane.

Pavement depths are required for all pavement rehabilitation projects. The typical test spacing is one core every 1 mile for each travel lane or shoulder to be tested and 1 per intersection. Project conditions must be evaluated to determine frequency of tests. Each core must be recorded on a Core/Exploration Hole Log sheet that includes the following information:

- Project name and State Route Number
- Location of the core, including the mile point, direction of travel, lane, and wheel path
- Core depth /Depth of Materials
- Depth of individual pavement lifts
- Description of the materials (core, base, subbase, subgrade), plasticity, moisture, Georgia soil classification (810.2), consistency or density
- If drilled on a crack, the type of crack (fatigue, transverse, etc.) and extent of crack
- Log must include a drawing showing the location of the core/hole in relation to pavement stripes and pavement edges

The Core Logs generated through this destructive test method should always be submitted with the design report. An example Pavement Design Core /Exploration Hole Log is provided at the end of this Appendix.

Exploration Holes

Exploration holes are used to gather information about underlying base materials and subgrade soils. Exploration holes must be used where needed to supplement as-constructed drawings for base depth, type, and quality and to obtain the necessary information about the materials to adequately characterize their properties for use in the design procedure. Base, soil, and other material samples can be obtained from exploration holes for laboratory testing. Groundwater levels should be measured and recorded when encountered.

Pavement depths, base thickness, subbase thickness, soil classification (GDOT-810.2 and AASHTO-M 145-91), and Soil Support Values are required for all pavement rehabilitation projects. Cores should be 6” to 8” in diameter to facilitate digging through the hole. The maximum core spacing is one every 1 mile for each travel lane or shoulder to be tested. Cores should be staggered such that the distance between cores is +/- ½ mile. Obtain Soil Support and 810.2 Series Samples (40 # bag) at the rate of 2 per mile, including both travel lanes. Conduct Resilient Modulus (M_R) tests (Cone Penetrometer Tests) in alternate core holes, 2-per mile. The minimum depth for soil exploration is typically 3-4 feet.

For 4 lane roads the Engineer should utilize his judgment in determining the distribution of tests in travel lanes. It would be advisable to reduce the overall number of cores, with accompanying tests, that would occur if the Destructive Test (DT) locations were simply doubled because it's a 4-lane road rather than a 2-lane road.

Remember, under Georgia Law a utility locate must be obtained at every location where an exploration hole is to be taken. Utility locates can be scheduled by calling the Utility Protection Center (770) 623-4344. You will need to provide the location for each proposed exploration hole.

Copies of Exploration Hole Logs (which can be included on the Core Logs discussed previously) and test results must be submitted with the Pavement Evaluation and design report as per the requirements outlined in the Deliverables section of this guide.

Field Testing for Misc. Special Circumstances

<u>Bridge Approaches for Major Projects</u>

Structures usually present grade control issues for paving projects. The situation is typically that we must maintain or reduce pavement grade at the bridges. Reducing grade normally occurs when asphalt concrete is to be removed from the bridge deck. The following minimum guidelines apply when testing at or near a structure:

- For structures with AC on the deck, at least one core is required at approximately the mid-span (through the AC only, do not core through the concrete deck).
- One core on each approach at approximately 10 ft from each end of the structure or approach slab.
- One additional core on each approach between 10 ft and 50 ft from each end of the structure or approach slab.
- Deflection testing at 5, 10, 20, 30, 40, 50, 75, 100, 125, 150, and 200 ft from each end of the structure.
- Do not core on a bare Portland Cement Concrete (PCC) deck.
- If an approach slab is present, measurements must be made from the end of the panel for the above testing locations. Do not core on an approach slab.

If the Bridge is to be replaced, the above testing is not required.

Laboratory Testing Guidelines

All tests would not be required on each project. The Engineer must utilize his judgment when deciding which tests are appropriate to a particular project. Examples of tests that could be required include, but are not limited to:

Asphalt Pavement

- Density of Cored Specimen
- Air Voids of Cored Specimen
- Specific Gravity of Cored Specimen; GDT-39
- Bitumen Extraction of Cored Specimen; GDT-37, GDT-83, GDT-125
- Mechanical Analysis of Extracted Aggregate; GDT-38
- Soil Classification; GDT 810.2, AASHTO M 145-91
- Soil Support Value
- Cement Penetration
- DSR – Dynamic Shear Rheometer
- Resilient Modulus; AASHTO T 307, NCHRP 1-28A
- Modulus of Elasticity
- Stripping (Lime Content)

Concrete Pavement

- ASR Test – Alkali Silica Reaction
- Tensile Splitting Strength; GDT-66
- Compressive Strength
- Modulus of Elasticity
- Soil Classification; GDT 810.2, AASHTO M 145-91
- Modulus of Subgrade Reaction

C.5 Pavement Design

Once the data collection, fieldwork and laboratory testing phases discussed in this Appendix are complete, a Pavement Design must be prepared for the project. The Designer should look at overlays, rehabilitation or complete reconstruction options for the project based on the findings from this evaluation, project specific requirements, pavement types available and formal design requirements are included in Chapter 11, as well as other chapters of this Design Manual.

In certain situations, as discussed in the Design Manual, Life Cycle Cost Analysis (LCCA) may be required before Pavement Design can be finalized and submitted for approval. The Designer should refer to the appropriate chapter of this manual for more details on LCCA.

C.6 Pavement Evaluation and Design Report

C.6.1 General

Pavement design recommendations and all supporting documentation including design assumptions, background information, and field data, must be compiled and submitted for review in a bound design report. The pavement design must be developed by, or under the direct supervision of, a Professional Civil Engineer registered in the State of Georgia. The engineer will place his/her Professional Seal on the pavement design report and will be the Engineer of Record for that design.

The design recommendations and supporting documentation shall be in either English or Metric units as specified in the contract documents. If no units are specified, English units shall be the primary unit of measurement.

The bound design report must include an executive summary and supporting documentation with contents as described in the next section.

C.6.2 Elements of a Pavement Evaluation and Design Report

C.6.2.1 Executive Summary

- Design Procedure and Design Life.
- Recommended pavement design(s) for all existing and new pavement features.
- Recommended Materials and Specifications, including:
  - Recommend materials to be used (reference applicable specification and bid item nomenclature)
  - For asphalt concrete pavement designs provide "Mix Design Level" for all layers proposed in accordance with the latest Superpave Mix Design Guidelines Letter, dated October 7, 2004, which can be found at: http://www.dot.state.ga.us/dot/preconstruction/consultantdesign/design/superpave.pdf
- Any required modifications to specifications, including required modifications to special provisions or specifications, and justification for this modification.

C.6.2.2 Supporting Documentation:

- Summary of historical “as-built” construction information, if available.
- Design Parameters, including:
  - A description of the project scope. Identify design procedures used and the design structural life for all new work and rehabilitation sections included in the report.
  - The basis for design parameters - design traffic, soil support value, subgrade modulus, regional factor, design reliability, and so on.
- Design calculations, including traffic, layer thickness, and total structure, etc.
- Design options and basis for recommendations.
- Pavement Condition Summary (PACES AND CPACES)
- An electronic copy of all raw deflection data files for the project (if applicable) shall also be provided on a 3½" floppy disk or CD-ROM.
- An electronic copy of all digital photograph files shall be provided on a 3½" floppy disk or CD-ROM.
- Life cycle cost calculation data (where applicable) – Where LCCA calculations are performed, supporting documentation for the input variables used (discount rate, analysis period, costs, activity timing) shall be provided. Where probabilistic LCCA is conducted, summary statistics of the results (min, max, mean, standard deviation) shall be presented along with histogram plots and cumulative distribution plots of Net Present Value (NPV) for each alternative.

Special Provisions

For projects that involve pavement rehabilitation, or construction of new pavement on portions of existing alignment, the report shall also include the following:

- Hard copy of deflection data - Deflections shall be shown for each sensor normalized to a 9,000 pound (4,082 Kg) load
- Plot of deflections by mile point or station
- Copies of all Core/Exploration Hole Logs
- A summary of all test results conducted on material samples
- Color copies or duplicates of all photos - Photos must be arranged in mile point order and labeled with the date, mile point and direction of the picture

- Summary of rut depth measurements - The summary must indicate the measured rut depths for each wheel track at each location. The average rut depth and standard deviation for each wheel track should also be indicated.

C.7 Deliverables Checklist

ITEM	YES	NO
Was the Preliminary Pavement Evaluation Report reviewed?		
Final Pavement Evaluation Report		
Was historical data gathered and analyzed? *Submit summary of data*		
Was traffic data obtained from(OEL)? *Identify alternate source.*		
Was the Concept Report reviewed?		
Was a PACES / CPACES analysis and report prepared? *Include report*		
Were photographs taken as required? *Include photographs*		
Was a detailed drainage survey performed? *Include report*		
Was Falling Weight Deflectometer (FWD) utilized? *Include normalized deflection analysis* *Include proof of Calibration*		
Was Ground Penetrating Radar (GPR) utilized? *Include test reports*		
Were Seismographic techniques utilized? *Include test results*		
Was an Inertial Profiler utilized? *Include test results*		
Was a dipstick or walking profiler utilized? *Include test results*		
Was destructive field testing utilized? *Include Core Logs* *Include Exploration Logs* *Include laboratory test results*		

ITEM	YES	NO
Were miscellaneous special conditions encountered? *Description of special circumstances and test results*		
Were alternative pavement designs prepared? *Submit pavement designs*		
Was Life Cycle Cost Analysis (LCCA) performed? *Submit LCCA study*		

TABLE C-4 DELIVERABLES CHECKLIST

C.8 Budget Workbook

PAVEMENT EVALUATON PERSONNEL MANHOUR ESTIMATE

$0

$0

NO.	TASK DESCRIPTION	PROJECT MANAGER	CHIEF or PRINCIPAL ENGINEER	PROJECT or SENIOR ENGINEER	FIELD ENGINEER/ GEOLOGIST	SENIOR TECH or Engr Aide	TECH	CADD TECH	CLERICAL	M.O.T. PERSONS	TASK TOTALS
I	**GENERAL**										
IA	SITE VISITS										0.0
IB	PROJECT MANAGEMENT, INVOICING, SCHEDULING & COORDINATING SUBCONTRACTORS, PREPARE: PROJECT MGT PLAN, QUALITY CONTROL PLAN, ORGANIZATIONAL PLAN, COMMUNICATION AND PROTOCOL PLAN										0.0
	PROJECT KICKOFF MEETING										
IC	PROJECT TEAM MEETINGS & CONSULTATION										0.0
ID	MUTCD PLAN PREPARATION / PERMIT										0.0
IE											0.0
II	**FIELD & OFFICE**										
IIA	PRELIMINARY PAVEMENT EVALUATION										0.0
IIB	UTILITY LOCATE										0.0
IIC	DISTRICT ENGINEER COORDINATION										0.0
IID	CORING LAYOUT (FIELD)										0.0
IIE	LOGGING/CLASSIFICATION & SUPERVISION OF CREWS										0.0
	CONE PENETROMETER TESTING-FIELD										
IIF	CORE HOLE BACKFILL										0.0
	UNSUITABLE PAVEMENT EVALUATION										
IIG	TRAVEL TO & FROM SITE for PERSONNEL										0.0
IIH	PAVEMENT EVALUATION - FIELD - PACES, OR CPACES										0.0
III	LABORATORY ASSIGNMENT,FIELD BOOK DATA REVIEW, PLAN PREPARATION										0.0
IIJ	RESEARCH PROJECT HISTORY										0.0
IIK	ENGINEERING ANALYSIS										0.0
IIL	PAVEMENT DESIGNS										0.0
IIM	REPORT PREPARATION										0.0
IIN	DRAFT REPORT PUBLISHING										0.0
IIO	REPORT REVISIONS										0.0
IIP	COMPILE FINAL REPORT										0.0
IIQ	QA/QC										0.0
IIR	MOT (2 PERSON CREW)										0.0
IIS											0.0
IIT											0.0
IIU											0.0
IIV											0.0
	PERSONNEL TOTALS	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**

PERSONNEL SUMMARY

PERSONNEL CLASSIFICATION	EST. HOURS	Avg. PAY RATE	COST($)		
PROJECT MANAGER	0.0		$0		
CHIEF or PRINCIPAL ENGINEER	0.0		$0		
PROJECT or SENIOR ENGINEER	0.0		$0		
FIELD ENGINEER/GEOLOGIST	0.0		$0		
ENGINEER'S AIDE/SENIOR TECHNICIAN	0.0		$0		
TECHNICIAN	0.0		$0		
CADD TECHNICIAN	0.0		$0		
CLERICAL	0.0		$0		
M.O.T. PERSONS	0.0		$0		
TOTAL HOURS	0.0	TOTAL COSTS			$0
OVERHEAD (INDIRECT COST ON LABOR ABOVE)					
LABOR X OVERHEAD RATE = OVERHEAD					$0
TOTAL DIRECT LABOR PLUS OVERHEAD					$0
	OTHER DIRECT COSTS (SPECIFY)				
MATERIALS & TRAVEL					
REPRODUCTION / COURIER			$0		
PERSONNEL PER DIEM (per person/day)			$0		
MILEAGE (per mile)			$0		
			$0		
			$0		
			$0		
TOTAL OTHER DIRECT COSTS					$0
TOTAL ESTIMATED COSTS					$0
PROFIT ON LABOR PLUS OVERHEAD				10.00%	$0
ESTIMATED COSTS (EXCLUDING FIELD & LAB)					$0

IID. CORING LAYOUT: 12 CORES @ 20 MIN/CORE = 4HRS

IIR. MOT-2 PERSONS 3 DAYS, 1 PERSON 3 DAYS =
2X3X8 = 48 HRS, 1X3X8 = 24 HRS, TOTAL = 72 HRS

0

C.9 Sample Pavement Evaluation Report

PAVEMENT EVALUATION SUMMARY

WRIGHTSBORO ROAD WIDENING AND REALIGNMENT

GDOT PROJECT NUMBER SPT-7001(9) RICHMOND COUNTY

PI NUMBER 250510

1. LOCATION / DESCRIPTION

This project is for the widening and realignment of Wrightsboro Road in Richmond County, Georgia. The project begins at Powell Road at station 34+06, extends to the east for approximately 2.5 miles, and ends at the entrance and exit ramps to I-520 (Bobby Jones Parkway) at station 168+90.

At the beginning of the project, between stations 34+06 to station 41+00, Wrightsboro Road is a 4-lane road with a center turn lane. At station 41+00, Wrightsboro Road narrows to a 2-lane county road. At station 154+00, Wrightsboro Road widens back to a 4-lane with a center turn lane, extending to project ending station 168+90 and beyond. The project is being designed as a 4-lane divided urban arterial, and is currently 2 lanes.

2. COPACES

A Copaces was not performed during this evaluation.

3. RUTTING

Rutting measurements taken at various locations along Wrightsboro Road averaged less than 1/8 inch in depth.

4. LOAD CRACKING

Level 1 and level 2 load cracking ranging between 20 and 100 percent was observed throughout much of the Wrightsboro Road Widening alignment. Occasional level 3 load cracking was observed, primarily near the beginning at the project at station 42+00, and also at station 16+00 of Belair Road. The following indicates the percentage and level of load cracking that was observed at each of the evaluated 100-foot sections.

	Load Cracking (%)		
Evaluated Test Section	**Level 1**	**Level 2**	**Level 3**
Wrightsboro Road (Inside West Bound Lane), Stations 39+00 to 40+00	20	0	0
Wrightsboro Road (Inside East Bound Lane), Stations 39+00 to 40+00	45	0	0
Wrightsboro Road, Stations 41+50 to 42+50	50	50	10
Wrightsboro Road, Stations 73+00 to 74+00	77	3	0
Wrightsboro Road, Stations 99+65 to 100+65	40	10	0
Wrightsboro Road, Stations 129+00 to 130+00	80	15	0
Wrightsboro Road, Stations 146+00 to147+00	30	0	0
Wrightsboro Road, Stations 167+00 to 168+00	10	0	0
Maddox Road, Stations 10+50 to 11+50	15	0	0
Flowing Wells Road, Stations 11+00 to 12+00	20	0	0
Belair Road, Stations 15+50 to 16+50	30	10	10

4. LOAD CRACKING (continued)

Evaluated Test Section	Load Cracking (%)		
	Level 1	Level 2	Level 3
Barton Chapel Road, Stations 44+00 to 45+00	40	0	0
Augusta W. Parkway, Stations 12+30 to 13+30	30	0	0

5. BLOCK CRACKING

There was little to no block/traverse cracking observed at the project beginning, between stations 37+50 (beginning of overlay) to 42+00. Block cracking ranging between 15 and 100 percent was observed along most of the Wrightsboro Road Widening alignment and each of the intersection roadway, (Maddox Road, Belair Road, Barton Chapel, etc.) beginning at station 42+00 where the current road narrows from 4 lanes to 2, and ending at station 168+00. The following indicates the percentage and level of block/traverse cracking that was observed at each of the evaluated 100-foot sections.

Evaluated Test Section	Block/Traverse Cracking (%)		
	Level 1	Level 2	Level 3
Wrightsboro Road (Inside West Bound Lane), Stations 39+00 to 40+00	0	0	0
Wrightsboro Road (Inside East Bound Lane), Stations 39+00 to 40+00	0	0	0
Wrightsboro Road, Stations 41+50 to 42+50	100	0	0
Wrightsboro Road, Stations 73+00 to 74+00	50	0	0
Wrightsboro Road, Stations 99+65 to 100+65	20	0	0
Wrightsboro Road, Stations 129+00 to 130+00	15	0	0
Wrightsboro Road, Stations 146+00 to147+00	0	0	0
Wrightsboro Road, Stations 167+00 to 168+00	25	0	0
Maddox Road, Stations 10+50 to 11+50	20	0	0
Flowing Wells Road, Stations 11+00 to 12+00	0	0	0
Belair Road, Stations 15+50 to 16+50	15	0	0
Barton Chapel Road, Stations 44+00 to 45+00	45	0	0
Augusta W. Parkway, Stations 12+30 to 13+30	45	0	0

6. REFLECTION CRACKING

No reflection cracking was observed

7. RAVELING

No raveling was observed along the alignment, with the exception being near station 100+00, which had less than 5 percent reveling.

8. EDGE DISTRESS

Typically, less than 5 percent of edge distress was observed along the Wrightsboro Road Widening alignment. However, approximately 33 percent of edge distress was observed between station 99+65 to 100+65, and 10 percent edge distress was observed at station 16+00 of Belair Road.

9. BLEEDING OR FLUSHING

Approximately 15 percent bleeding/flushing was observed between stations 129+00 to 130+00.

10. CORRUGATION OR PUSHING

No corrugation or pushing was observed

11. LOSS OF SECTION

No loss of section was observed.

12. CORES

Core samples were taken at ten different locations. Depths of the existing asphaltic concrete pavement varied from 6 to 15 ½ inches thick. Base material typically consisted of silty/clayey Sand.

Core Number 1
Location: This core was obtained at the inner eastbound lane, just prior to Wrightsboro narrowing down to 2 lanes. (Station 39+00)
Asphalt Thickness: 15 ½
Base Material Type Silty Sand Base
Curb and Gutter Section

Core Number 2
Location: This core was obtained from the west bound land at the intersection with Maddox Road. (Station 52+00)
Asphalt Thickness: 6
Base Material Type Silty Sand Base
Drainage ditches with no curbs.

13. CORES (CONT'D)

Core Number 3
Location: This core was obtained from the east bound land at the intersection with Lukes Road. (Station 65+40)
Asphalt Thickness: 9
Base Material Type Silty Sand Base
Drainage ditches with no curbs.

Core Number 4
Location: This core was obtained from the west bound land at the intersection with Flowing Wells Road. (Station 93+63)
Asphalt Thickness: 6 ½
Base Material Type Clayey Sand Base
Drainage ditches with no curbs.

Core Number 5
Location: This core was obtained from the east bound land at station 118+20, near the beginning of roadway realignment. (Station 118+20)
Asphalt Thickness: 6 ½
Base Material Type Clayey Sand Base
Drainage ditches with no curbs.

Core Number 6
Location: This core was obtained from the west bound land at the intersection with Belair Road. (Station 144+60)
Asphalt Thickness: 6
Base Material Type Clayey Sand Base
Drainage ditches with no curbs.

Core Number 7
Location: This core was obtained from the outer east bound land at the intersection with Barton Chapel Road. (Station 155+00)
Asphalt Thickness: 14 ¼
Base Material Type Silty Sand Base
Curb and Gutter Section

13. CORES (CONT'D)

Core Number 8
Location: This core was obtained from the outer west bound land at the intersection with Crescent Drive. (Station 159+00)
Asphalt Thickness: 10 ¾
Base Material Type Silty Sand Base
Curb and Gutter Section

Core Number 9
Location: This core was obtained from the west bound land at the intersection with Augusta West Parkway. (Station 161+72)
Asphalt Thickness: 9
Base Material Type Silty Sand Base
Curb and Gutter Section

Core Number 10
Location: This core was obtained from the outer west bound lane near the end of the project, at the I-520 exit ramp. (Station 168+50).
Asphalt Thickness: 10
Base Material Type Silty Sand Base
Curb and Gutter Section

14. Pavement Condition Summary

From station 34+06 to station 41+00, the existing pavement of Wrightsboro Road is in good condition. Partial milling (1½ -inches) of the surface with an overlay is recommended.

From station 41+00 to station 154+00, the existing pavement along this 2-lane section of Wrightsboro Road is in fair condition. Full Depth reconstruction is recommended based on the assessed condition and because of street realignment and median construction, which will occur over a large portion of the alignment.

From station 154+00 to station 168+00, the existing pavement along this 4-lane section of Wrightsboro Road is also in fair condition. Stripping of the asphalt was observed in the cores obtained from this section of roadway. Full depth reconstruction is also recommended along this section of roadway alignment due to the current pavement condition and current under design based on traffic projections.

15. Full Depth Sections

The following full depth options are presented for this project:

Alternate 1:
It is recommended that the flexible pavement for station 41+00 to station 168+00 of Wrightsboro Road be reconstructed/constructed full depth utilizing asphaltic concrete with a graded aggregate base.

Alternate 2:
As an alternate, a full depth flexible pavement utilizing cement stabilized reclaimed base is included for station 41+00 to station 168+00. Special Provision Section 301, attached should be adhered to for this alternate.

16. Overlay Sections

An overlay design specific to stations 34+06 to station 41+00 is included.

17. Other

New pavements should be constructed flush with all existing and/or new utility manholes or vaults.

Remove all paint and markers before overlays.

February 1, 2005

Reported by: Dana R. Causby, P.E.

Reviewed by: Randall L Bagwell, P.E.

SPECIAL PROVISIONS

February 1, 2005

Georgia Department of Transportation
State of Georgia

Special Provision

Section 301—Soil Cement

Retain Section 301 and add the following:

Section 301—Cement Stabilized Reclaimed Base Construction

301.1 General Description

This work includes constructing a cement stabilized base course by pulverizing the existing pavement structure and mixing with Portland cement to the depth specified on the plans. Construct according to these Specifications and to the lines, grades, thickness, and typical cross-sections shown on the Plans or established by the Engineer.

301.1.01 Related References

A. Standard Specifications

Section 412—Bituminous Prime

Section 800—Coarse Aggregate

Section 814—Soil Base Materials

Section 821—Cutback Asphalt

Section 830—Portland Cement

Section 880—Water

B. Referenced Documents

General Provisions 101 through 150

GDT Test Methods			
GDT 19	GDT 21	GDT 65	GDT 86
GDT 20	GDT 59	GDT 67	

301.1.01 Submittals

Before constructing a test section according to Subsection 301.3.04.E.1, submit a Construction Work Plan to the Engineer. Include proposed equipment and proposed compaction procedures. If the Engineer determines that the Work Plan is not satisfactory, revise the compaction procedure and augment or replace equipment, as necessary, to complete the Work.

301.2 Materials

Ensure that materials meet the requirements of the following Specifications:

Material	Section
Blotter material (sand)	412.3.05.G.3
Coarse Aggregate	800
Soil Base Material	814.2.02
Cutback asphalt, RC-30, RC-70, RC-250 or MC-30, MC-70, MC-250	821.2.01
Portland Cement (Type I or Type II)	830.2.01
Water	880.2.01

SPECIAL PROVISION SECTION 301
CEMENT STABILIZED RECLAIMED BASE CONSTRUCTION

301.3 Construction Requirements

301.3.01 Personnel

Ensure that only experienced and capable personnel operate equipment.

301.3.02 Equipment

Use equipment that has been approved by the Engineer before construction begins. Provide equipment in satisfactory condition capable of continuously mixing materials (pavement structure, soil, water, and cement) to a consistent depth. Use equipment capable of providing a homogenous blend.

301.3.03 Preparation

Loosen and pulverize the in-place pavement structure to the width and depth to be stabilized without damaging the underlying materials. Add water to assist pulverization if necessary.

301.3.04 Construction

A. Weather Limitations

1. Mix cement-stabilized base only when the weather permits the course to be finished without interruption within the time specified.
2. Mix materials only when the moisture of the materials to be used in the mixture meets the specified limits.
3. Begin mixing only when the air temperature is above 40^{o}F (4^{o}C) in the shade and rising.
4. Ensure that the temperature of the pavement course and underlying materials are above 50^{o}F (10^{o}C).
5. If the work is interrupted for more than two hours after cement has been added, or if rain increases the cement's moisture content outside the specified limits, remove and replace the affected portion at no additional cost to the Department.

B. Moisture Adjustment

Adjust the moisture content of the roadway materials to within 100 to 120 percent of the optimum moisture immediately before spreading the cement. The optimum moisture content is determined by the Job Mix Design and can be adjusted by the Engineer.

C. Cement Application

1. Uniformly spread the required amount of Portland cement with a cyclone-type mechanical spreader or its equivalent. Do not use pneumatic tubes to transfer the cement from the tanker directly onto the material to be stabilized.
2. Apply cement at the rate specified on the Job Mix Design (as established by GDT-65) and mix to the depth shown on the Plans. The Engineer may alter the spread rate during the progress of construction if necessary. Maintain the application rate within ± 10 percent of that specified by the Engineer.
3. Provide both equipment and personnel to measure the application rate of cement placed.
4. Apply cement on days when wind will not interfere with spreading.
5. If the cement content is below the 10 percent limit in the mixing area, add additional cement to bring the affected area within the tolerance specified and recalibrate the mechanical spreader's spread rate. If the cement content is more than the 10 percent limit in the mixing area, the excess quantity will be deducted from the Contractor's pay for cement.
6. Regulate operations to limit the application of cement to sections small enough so that all of the mixing, compacting, and finishing operations can be completed within the required time limits.
7. Pass only spreading and mixing equipment over the spread cement and operate this equipment so that it does not displace cement.

8. Replace damaged cement at no cost to the Department when damage is caused by:
   - Hydration due to rain, before or during mixing operations.
   - Spreading procedures contrary to the requirements stated above.
   - Displacement by the Contractor’s equipment or other traffic.
9. Do not spread cement on any areas that “pump” under construction traffic.

D. Mixing

1. Begin mixing as soon as possible after the cement is spread, and continue until a homogeneous and uniform mixture is produced. Make any necessary changes to meet the Engineer’s requirements if the equipment does not produce a homogeneous and uniform mixture conforming to these Specifications.
2. Continue pulverizing until the base mixture is uniform in color and conforms to the following gradation requirements
   - 95 percent passing the 2 inch (50mm) sieve
   - 55 percent of the roadway material, excluding gravel, passes the No. 4 (4.75mm) sieve.
3. Add water as needed to maintain or bring the moisture content to within the moisture requirements immediately after the preliminary mixing of the cement and roadway material.
4. Mix the additional water homogeneously into the full depth of the mixture.

E. Compaction and Finishing

1. Test Section
   a. Use the first section of each constructed cement-stabilized base course as a test section.
   b. Construct a test section between 350 feet (100m) and 500 feet (150m) long at the designated width.
   c. The Engineer will evaluate compaction, moisture, homogeneity of mixture, thickness of stabilization, and finished base surface. If the Engineer deems necessary, revise the compaction procedure or augment or replace equipment.
2. Time Limits
   a. Begin compaction within 45 minutes from the time water is added to the cement mixture.
   b. Complete compaction within 2 hours.
   c. Complete all operations within 4 hours, from adding cement to finishing the surface.
   d. Do not perform vibratory compaction on materials more than 90 minutes old, measured from the time cement was added to the mixture.
3. Moisture Control

   During compaction, ensure that the moisture is uniformly distributed throughout the mixture at a level of between 100 and 120 percent of the optimum moisture content.
4. Compaction Requirements
   a. Use a sheep’s foot or steel wheel roller for initial compactive effort unless an alternate method is approved by the Engineer.
   b. Compact the cement-stabilized base course to at least 98 percent of the maximum dry density established on the Job Mix Design.
   c. Uniformly compact the mixture and then shape to the grade, line, and cross- section shown on the Plans.
   d. Remove all loosened material accumulated during the shaping process. Do not use additional layers of cement-treated materials in order to conform to cross-sectional or grade requirements.
   e. Use a pneumatic-tired roller to roll the finished surface until it is smooth, closely knit, and free from cracks or deformations, and conforming to the proper line, grade, and cross-section.

f. In places inaccessible to the roller, obtain the required compaction with mechanical tampers approved by the Engineer. Apply the same compaction requirements as stated above in Subsection 301.3.04.E.4.

g. Perform grading operations immediately after the placement and compaction operations. Roll the stabilized base course again with a pneumatic-tired roller.

F. Construction Joints

1. Form a straight transverse joint at the end of each day's construction or whenever the Work is interrupted.
2. Create the straight transverse joint by cutting back into the completed Work to form a true vertical face free of loose or shattered material.
3. Form the joint at least 2 feet (600mm) from the point where the spreader strike-off plate comes to rest at the end of the day's work, or at the point of interruption.
4. Form a longitudinal joint as described above if cement-stabilized mixture is placed over a large area where it is impractical to complete the full width during one day's work. Use the procedure for forming a straight transverse joint. Remove all waste material from the compacted base.

G. Priming the Base

1. Apply bituminous prime according to Section 412 as soon as possible and in no case later than 24 hours after completion of the finishing operations.
2. Apply prime only to an entirely moist surface. If weather delays prime application, apply prime as soon as the surface moisture is adequate.
3. Maintain and protect the curing seal for seven days.
4. Protect finished portions of the cement-stabilized base course that are used by equipment in the construction of an adjoining section to prevent marring or damaging of the completed Work. Protect the stabilized area from freezing during the curing period.

H. Opening to Traffic

1. Do not permit any traffic or equipment on the finished surface of the base course until the prime has hardened enough so that it does not pick up under traffic. For the first seven days after priming, restrict traffic to lightweight vehicles such as passenger cars and pickup trucks. Do not allow vehicles with an average axle load exceeding 20,000 pounds (9Mg) on the unfinished base at any time.
2. Correct any failures caused by traffic at no additional cost to the Department.

I. Protection of Course

Maintain the base course until the Engineer determines that it has sufficiently cured and is ready to be covered with the pavement course. Make repairs specified in Subsection 300.3.06.B, whenever defects appear. This preservation action does not relieve the Contractor of his responsibility to maintain the Work until final acceptance as specified in Section 105.

301.3.06 Quality Acceptance

A. Compaction Tests

1. Determine the maximum dry density from representative samples of compacted material, according to GDT 19 or GDT 67.
2. Determine the in-place density of finished courses according to GDT 20, GDT 21 or GDT 59 , as soon as possible after compaction, but before the cement sets.

B. Gradation Test

Ensure that the gradation of the completely mixed cement-stabilized base course meets the requirements of Subsection 301.3.04.D.2.

C. Finished Surface

Check the finished surface of the cement-stabilized base course transversely.

1. Check the surface using a 15 ft (4.5 m) straightedge parallel to the centerline.

   Additionally, use one of the following tools:

   - A template, cut true to the required cross-section and set with a spirit level on non-superelevated sections

- A system of ordinates, measured from a stringline
- A surveyor's level

2. Ensure that ordinates measured from the bottom of the template, stringline, or straightedge, to the surface do not exceed 1/4 in (6 mm) at any point. Rod readings shall not deviate more than 0.02 ft (6 mm) from required readings.
3. Correct any variations from these requirements immediately according to Subsection 300.3.06.B, "Repairing Defects."

C. Thickness Tolerances

1. Thickness Measurements

   Determine the thickness of the cement-stabilized base course, by making as many checks as necessary to determine the average thickness, but not less than one check per 1000 feet (300m) per 2 lanes.

2. Excess Thickness

   a. Determine the average thickness per linear mile (kilometer) from all measurements within each mile (kilometer) increment.

   b. Ensure that the average thickness does not exceed the specified thickness by more than ½ in (13 mm).

   c. If the basis of payment is per cubic yard (meter), and the average thickness for any mile (kilometer) increment exceeds the allowable ½ in (13 mm) tolerance, the excess quantity in that increment will be deducted from the Contractor's payments.

   d. The excess quantity is calculated by multiplying the average thickness that exceeds the allowable ½ in (13 mm) tolerance by the surface area of the base, as applicable.

E. Strength

1. Ensure that the strength of the completed cement-stabilized base course is at least 300psi (2070kPa), as determined from testing the unconfined compressive strength of cores from the completed course in accordance with GDT 86.

2. If a strength test falls below 300psi (2070kPa), do the following:

   a. Isolate the affected areas by securing additional cores every 75 feet (23m) on each side of the failing area.

   b. Average all compressive strengths in the affected area to determine the basis for corrective work according to the following table or the Engineer's directions.

Compressive Strength	Corrective Work
300 psi (2070 kPa) or greater	None
200 psi (1380 kPa) to 299 psi (2069 kPa)	6" & 8" (150mm & 200mm) base – add 135lbs/yd^2 (75kg/m^2) asphaltic concrete
Less than 200 psi (1380kPa)	Reconstruct affected area
Notes: 1) *Ensure that a corrected area requiring asphaltic concrete is at least 150ft (45m) long and covers the full width of the cement-stabilized base surface.* 2) Perform corrective work requiring asphaltic concrete or reconstruction at no additional cost to the Department.	

301.4 Measurement

A. Base Material

Measure base material by the cubic yard (meter), loose volume, as specified in Section 109, during mixed-in–place construction when it is necessary to add materials to the roadbed or to build up the base with new material.

B. Cement-Stabilized Base Course

Measure the surface length along the centerline when payment is specified by the square yard (meter). The width is specified on the Plans. Measure irregular areas, such as turnouts and intersections, by the square yard (meter).

C. Portland Cement

Measure Portland cement by the ton (megagram).

D. Bituminous Prime

Bituminous prime is not measured for separate payment. Include the cost of furnishing and applying bituminous prime according to the provisions of Section 412 in the Unit Price Bid for each individual base item.

E. Coarse Aggregate

Measure coarse aggregate by the ton (megagram).

301.5 Payment

A. Base Material

When it is necessary to add other materials to those in the roadbed, or to build up the base with entirely new materials, the added base materials, will be paid for at the Contract Unit Price per square yard (meter), complete, in place, and accepted. Payment will be full compensation for soil-cement material, mixing in the pit, loading, unloading, and spreading.

B. Cement-Stabilized Base Course

Cement-stabilized base, in-place and accepted, will be paid for at the Contract Unit Price per square yard (meter). Payment will be full compensation for roadbed preparation, mixing on the road, shaping, pulverizing, watering, compaction, defect repair, and maintenance.

C. Portland Cement

Portland cement will be paid for at the Contract Unit Price per ton (megagram). Payment is full compensation for furnishing, hauling, and applying the material. Only Type I or Type II Portland cement incorporated into the finished course will be paid for and no payment will be made for cement used to correct defects due to the Contractor’s negligence, faulty equipment, or error.

D. Coarse Aggregate

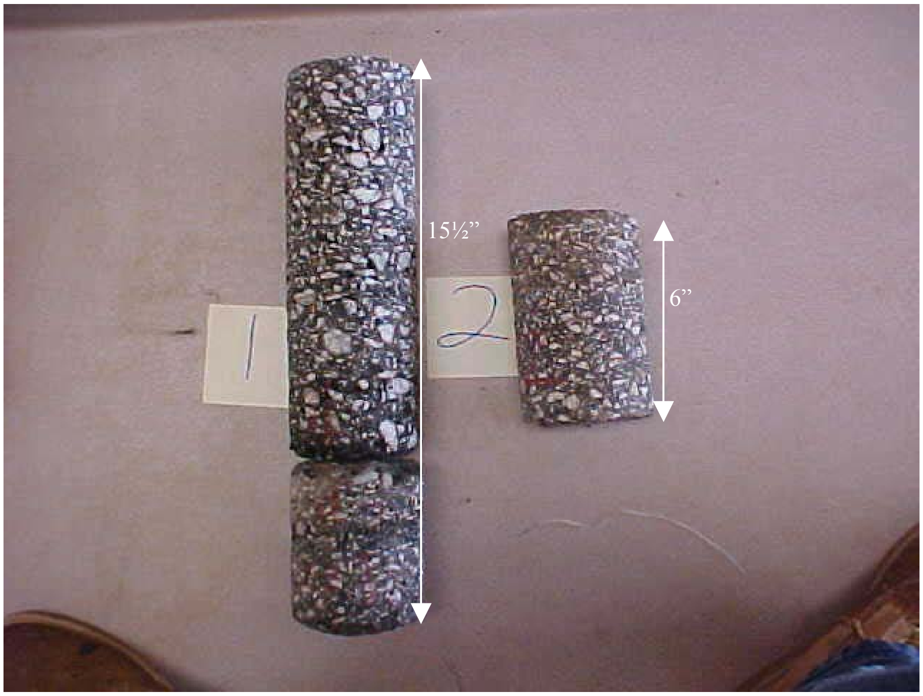
Coarse aggregate will be paid for at the Contract Unit Price per ton (megagram). Payment is full compensation for furnishing, hauling, spreading, watering, shaping, and compacting the material.

Payment will be made under:

Item No. 301	Base—including material	Per cubic yard (meter)
Item No. 301	Cement Treated Base Course	Per square yard (meter)
Item No. 301	Type I or Type II Portland Cement	Per ton (megagram)
Item No. 800	Coarse Aggregate – including material	Per ton (megagram)

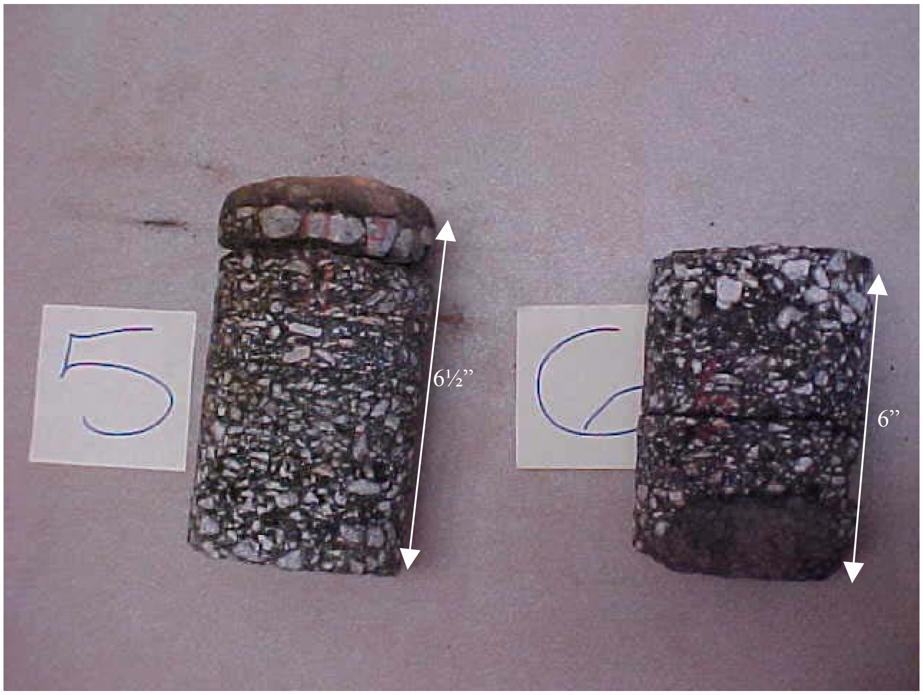
PHOTOGRAPHS

June 11, 2004



Core #1: Station 39+00
Core #2: Station 52+00

Core #3: Station 65+40
Core #4: Station 93+63



Core #5: Station 118+20
Core #6: Station144+60

Core #7: Station 155+00
Core #8: Station 159+00

Core #9: Station 161+72
Core #10: Station 168+50

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: SPT 7001(9) **County:** Richmond
P.I. no.: 250510
Description: Wrightsboro Road Widening & Realignment

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 3.00%
AADT initial year of design period: 16,200 vpd (2004)
AADT final year of design period: 21,600 vpd (2025)
Mean AADT (one-way): 18,900 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
18,900	*	0.80	*	0.030	*	1.17	=	532

Total predicted design period loading = 532 * 21 * 365 = 4,077,780

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 2.50
Regional Factor: 1.50

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
12.5 mm Superpave	1.50	(38)	0.44	0.66
19 mm Superpave	3.00	(76)	0.44	1.32
25 mm Superpave	0.00	()	0.44	0.00
	7.00	(178)	0.30	2.10
Graded Aggregate Base	10.00	(254)	0.16	1.60

Required SN = 5.48 Proposed SN = 5.68

>>> Proposed pavement is 3.6% Overdesign <<<

Remarks:

Prepared by Randall L. Bagwell, P.E. (NOVA Engineering)
June 10, 2004
Date

Recommended ____________________
State Urban Design Engineer **Date**

Approved ____________________
State Pavement Engineer **Date**

D Appendix D – Structural Coefficients

MATERIAL	COEFFICIENT
Asphaltic Concrete (Top 4 1/2")	0.44
Asphaltic Concrete (>4 1/2" from surface)	0.30
Calcium Chloride Stabilized Limestone Base	0.18
Cement Stabilized Chert Base	0.20
Cement Stabilized Graded Aggregate	0.22
Graded Aggregate and Crushed Limestone (Compacted to Modified Density)	0.16
Graded Aggregate and Crushed Limestone (Compacted to 96% of Modified Density)	0.14
Limerock Base (Compacted to Modified Density)	0.16
Limerock Base (Compacted to 96% of Modified Density)	0.12
Sand Asphalt	0.18
Sand Bituminous Stabilized Base (6")	0.12
Soil Aggregate Base	0.12
Soil Cement	0.20
Surface Treatment (Triple)	0.20
Sand Clay Base	0.10
For Overlay Design:	
Old Asphaltic Concrete	0.30
Old Portland Cement Concrete	0.40

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: SPT 7001(9) **County:** Richmond
P.I. no.: 250510
Description: Wrightsboro Road Widening & Realignment

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 3.00%
AADT initial year of design period: 16,200 vpd (2004)
AADT final year of design period: 21,600 vpd (2025)
Mean AADT (one-way): 18,900 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
18,900	*	0.80	*	0.030	*	1.17	=	532

Total predicted design period loading = 532 * 21 * 365 = 4,077,780

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 2.50
Regional Factor: 1.50

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
12.5 mm Superpave	1.50	(38)	0.44	0.66
19 mm Superpave	3.00	(76)	0.44	1.32
	1.00	(25)	0.30	0.30
25 mm Superpave	10.00	(254)	0.30	3.00

Required SN = 5.48 Proposed SN = 5.28

>>> Proposed pavement is 3.7% Underdesign <<<

Remarks:

Prepared by Randall L. Bagwell, P.E. (NOVA Engineering)
June 10, 2004
Date

Recommended ____________________
State Urban Design Engineer **Date**

Approved ____________________
State Pavement Engineer **Date**

FLEXIBLE PAVEMENT DESIGN ANALYSIS

Project: SPT 7001(9) **County:** Richmond
P.I. no.: 250510
Description: Wrightsboro Road Widening & Realignment

Traffic Data (NOTE: AADTs are one-way)
24-hour Truck Percentage: 3.00%
AADT initial year of design period: 16,200 vpd (2004)
AADT final year of design period: 21,600 vpd (2025)
Mean AADT (one-way): 18,900 vpd

Design Loading

Mean AADT		LDF		Trucks		18-K ESAL		Total Daily Loads
18,900	*	0.80	*	0.030	*	1.17	=	532

Total predicted design period loading = 532 * 21 * 365 = 4,077,780

Design Data
Terminal Serviceability Index: 2.50
Soil Support: 2.50
Regional Factor: 1.50

PROPOSED FLEXIBLE PAVEMENT STRUCTURE

Material	Thickness Inches	(mm)	Structural Coefficient	Structural Value
	*** OVERLAY ***			
12.5 mm Superpave	1.50	(38)	0.44	0.66
	*** EXISTING PAVEMENT ***			
Asphaltic Concrete	3.00	(76)	0.44	1.32
	10.50	(267)	0.30	3.15
Sand-Clay Base	6.00	(152)	0.10	0.60

Required SN = 5.48 Proposed SN = 5.73

>>> Proposed pavement is 4.5% Overdesign <<<

Remarks:

Prepared by Randall L. Bagwell, P.E. (NOVA Engineering)
June 10, 2004
Date

Recommended ______________________________
State Urban Design Engineer **Date**

Approved ______________________________
State Pavement Engineer **Date**

Georgia Department of Transportation

Office of Maintenance

PACES

PAvement Condition Evaluation System

2004

Pavement Condition Evaluation System

Table of Contents

APPENDIX E

Chapter I Basics of the System

Introduction

The Pavement Condition Evaluation System (PACES) is designed to indicate the amount and type of surface distress on a roadway at the time the survey is made. The system standardizes the terminology for the types of defects that can be found on a pavement in Georgia and defines the various levels of severity for these defects. This system will allow roads to be rated objectively statewide.

This system only addresses the structural condition of the pavement surface. It does not include skid resistance and ride-ability because these will be measured with high speed testing equipment.

General Outline

A number of distresses have been identified for flexible pavement and surface treatment which relate to the performance of the pavement. Both the presence of these distresses and the severity levels must be taken into account when rating a pavement. These distresses are as follows (also see distress definitions):

Rut Depth	*Raveling*
Load Cracking	*Edge Distress*
Block Cracking	*Bleeding/Flushing*
Reflection Cracking	*Corrugations/Pushing*
Patches and Potholes	*Loss of Section*

There are other types of defects which are not being considered either because they occur infrequently or they are included in one of the above categories at a certain severity level. Transverse cracking for instance is considered to be an initial stage of block cracking and is therefore rated in that category.

Ratings are done for each mile (or partial mile) by selecting a sample section for cracking distresses representative of the pavement condition for that rating segment. The defects noted for each rating segment within a project are then averaged to obtain the representative pavement condition for that project. A project rating is determined from deduct values which have been established for each defect and severity level.

APPENDIX E

Flexible Pavement Distress Definitions

The various pavement distresses are defined in this section along with descriptions and illustrations of the various levels of severity for each distress. The rater must be thoroughly familiar with the distresses and severity levels as defined in this section. The rater may or may not agree with all of the definitions and descriptions presented in this section, but all pavement sections must be rated in accordance with the criteria presented here in order for the rating system to be uniform.

Illustrations and photographs are shown for the various distresses and the severity levels which represent what typically might be seen in the field. The illustrations do not show all conditions that might be found nor is it intended that a condition must look exactly like what is shown in this manual for it to be rated at a particular severity level. The pictures are simple illustrations of what the rater is likely to see for a certain distress at a certain severity level. The rater must use his judgment based on the descriptions and the pictures while classifying the distress and severity level found on a sample section. It is possible that a combination of distresses and severity levels are present and these combinations should be recorded on the survey sheet as they exist.

Rut Depth

Definition:

Rutting is longitudinal depressions that form under traffic in the wheelpaths and are greater than 20 feet long. Rutting is a permanent deformation of the wheelpaths caused by traffic loadings. Rutting can be caused by insufficient compaction, plastic movement of the mix, or an unstable foundation.

According to its definition, the following questions may be addressed when identifying rutting distress.

1. Compared with the pavement outside the wheelpaths, is there any depression deformation in the wheelpaths?

2. Is this deformation in the longitudinal direction?

3. Is there any cracking in the longitudinal direction which is associated with the deformation in the wheelpaths?

How to Measure:

Rut Depth will be estimated in both wheelpaths in the sample area and recorded on the survey in units of 1/8 of an inch. If rutting is extensive (more than 3/8 inch), actual measurement may be necessary.

Severity levels for Rutting are not applicable.

Load Cracking

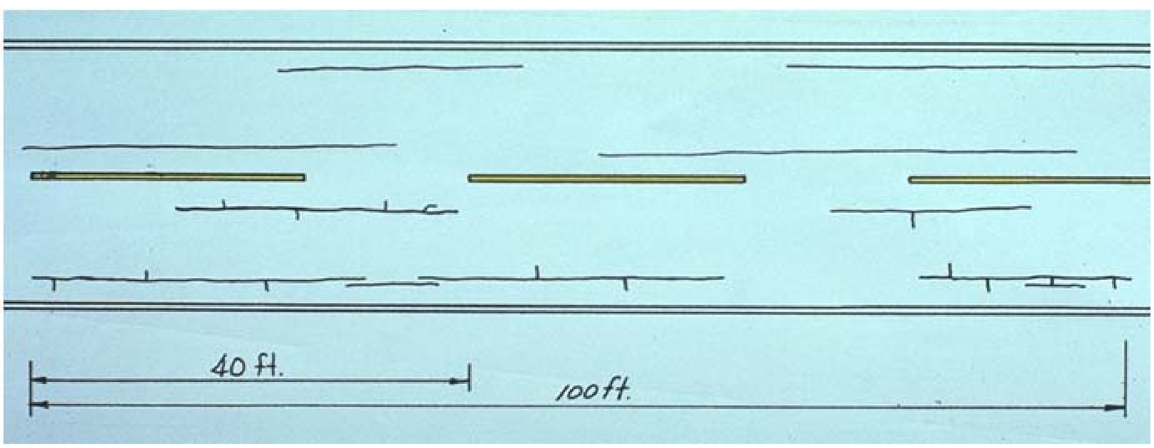
Description:

This type of cracking is caused by repeated heavy loads and always occurs in the wheelpaths. This type cracking usually starts as single longitudinal cracks in the wheelpaths. As progression continues, short transverse cracks occur that intersect the original longitudinal cracks. Additional longitudinal cracks occur in the wheelpaths. As the number of longitudinal and transverse cracks in the wheelpaths increases, polygons are formed by the intersection of these cracks. As deterioration continues, these polygons become smaller (due to additional cracking) and, in the worse case, begin to pop out. When load cracking progresses to the point where small polygons are formed, rutting can become extensive and pumping of base material can occur.

Following are examples for each severity level of load cracking.

Load Cracking **(Severity Level 1)**

Level 1 Load Crack patterns are generally tight single longitudinal cracks in the wheelpaths. A wheelpath is approximately 3 feet wide and load cracking can occur at the edge of the wheelpath. Occasional short, tight longitudinal cracks parallel to the main longitudinal cracks can also occur and still be defined as level 1 load cracking pattern.

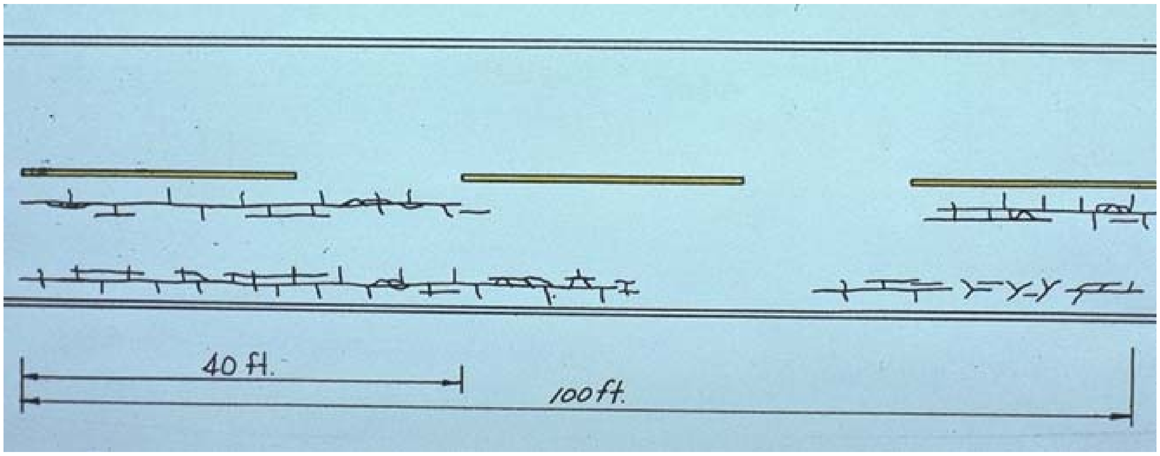


The illustration above is an example of the range in load cracking patterns to be recorded as level 1 load cracking. There is approximately 120 feet of cracking in the two wheelpaths or 60% of the sample in lane one in the two wheelpaths or 60% of the sample in lane one and 130 feet (65%) of level 1 load cracking in lane two.

Examples of Level 1 Load Cracking

Load Cracking **(Severity Level 2)**

The following illustration shows the general range in appearance of level 2 load cracking patterns. These cracks are wider than level 1cracks and occur only in the wheelpaths. This level cracking has a single or double longitudinal crack with a much larger number of 0-2 ft. transverse cracks intersecting than in level 1 load cracking. Occasionally polygons will form, but are not prominent.



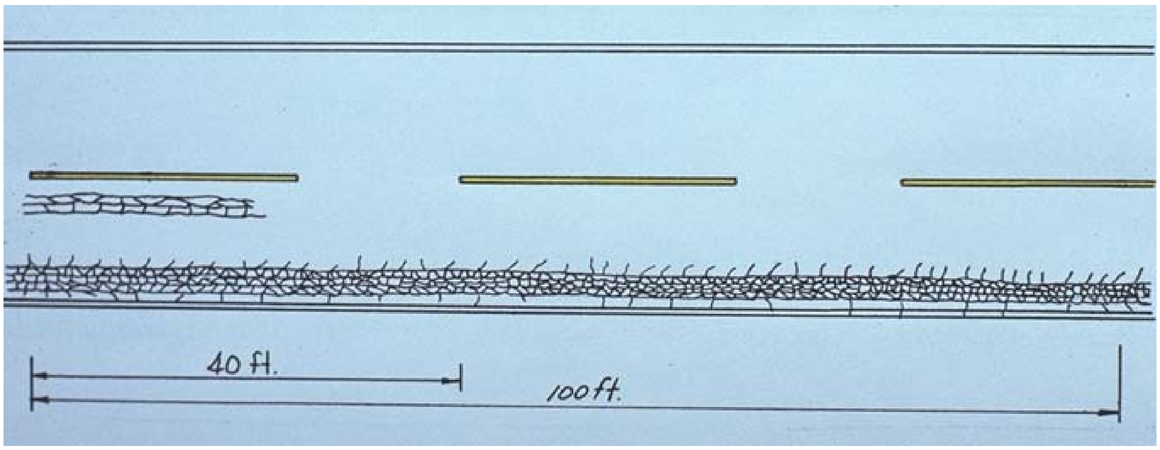
In this example, there is approximately 150 ft. of cracking in the 100 ft. sample area, or 75% of sample area.

APPENDIX E

Examples of Level 2 Load Cracking

Load Cracking **(Severity Level 3)**

The illustration shows the general appearance of level 3 load cracking patterns. This type pattern generally has three or more longitudinal cracks in the wheelpaths with many interconnecting transverse cracks. Many small polygons are formed causing the appearance of “alligator hide”. This type cracking is marked by a definite, extensive pattern of small polygons and is sometimes accompanied by severe rutting.



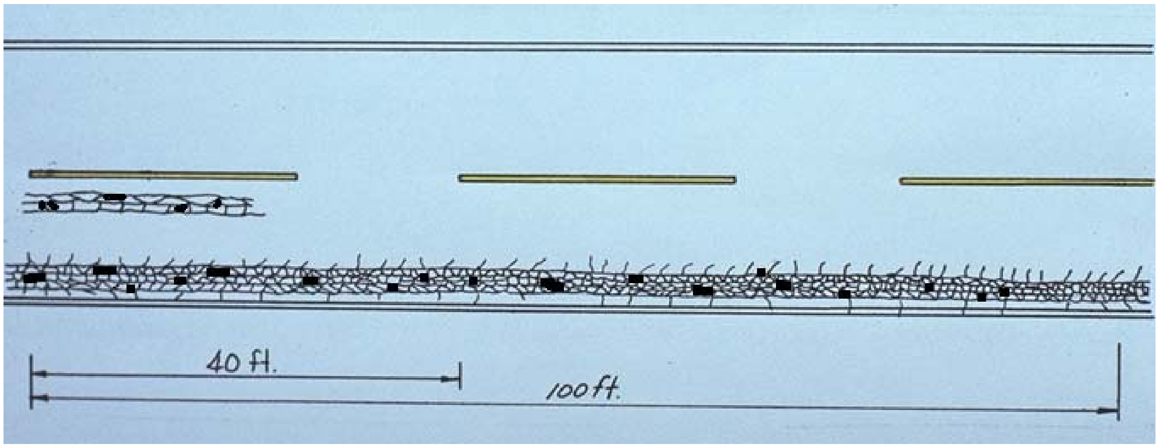
In this example, 60% of the sample has level 3 load cracking.

APPENDIX E

Examples of Level 3 Load Cracking

Load Cracking **(Severity Level 4)**

The following illustration shows level 4 load cracking patterns. This type pattern has the definite "Alligator hide" pattern, but had deteriorated to the point that the small polygons are beginning to pop out. Rutting is usually severe and pumping of base material is sometimes evident.



In this example, 60% of the sample area has level 4 load cracking.

Examples of Level 4 Load Cracking

Block/Transverse Cracking

This type cracking is caused by weathering of the pavement or shrinkage of cement treated base materials. Block/transverse cracking is not load related. The block pattern is distributed uniformly throughout the roadway and not concentrated in the wheelpaths. Block cracking is interconnecting cracks forming a series of large blocks usually with sharp corners.

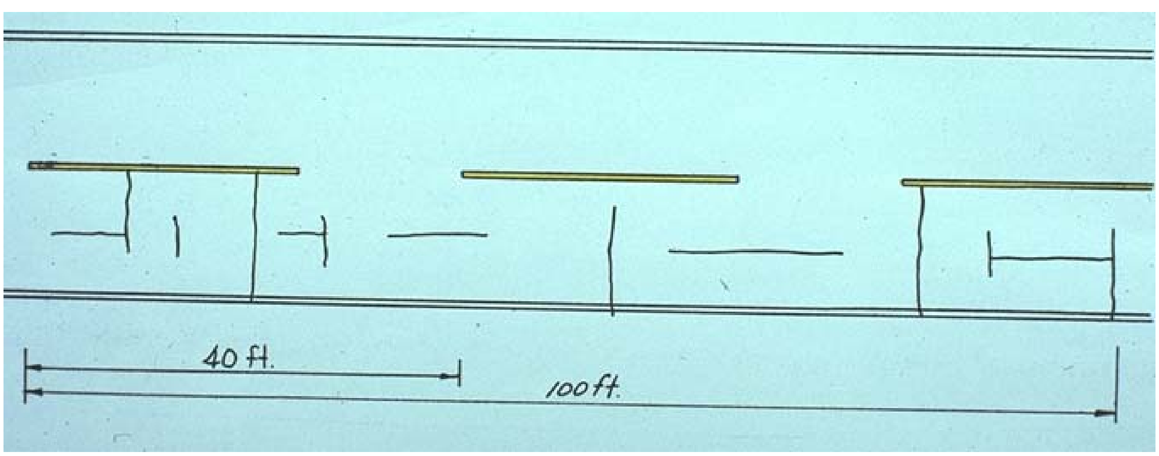
Block/transverse cracking begins as single, tight transverse, longitudinal or combinations of both types of cracks. In the beginning, block/transverse cracks may not form a recognizable block pattern, just longitudinal and/or transverse cracks that are not associated with the wheelpaths.

As this type of cracking progresses, a definite block pattern occurs and the cracks become wider. As the cracking becomes worse, the block pattern densifies (small blocks) and/or the cracks become very wide (> 1/8 inch).

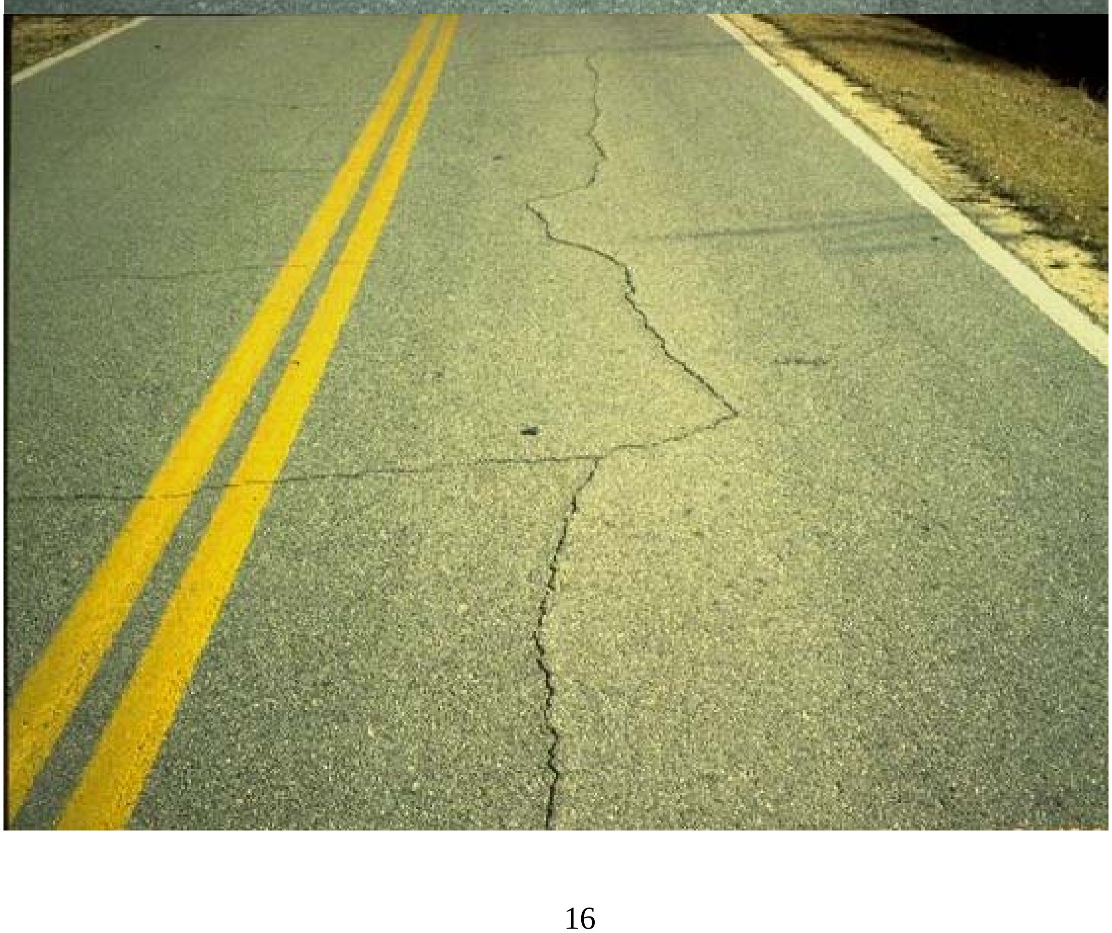
Severity Level 1 Block/Transverse Cracking

This type cracking is not load related and does not occur in the wheelpaths. Level 1 block/transverse cracking is made up of transverse, longitudinal, or a combination of both types of cracks. A definite "block" pattern has not developed yet. The longitudinal cracks are tight and not in the wheelpaths although they may wander into the wheelpaths at times.

The illustration below would be considered to have level 1 block/transverse cracking on 100 percent of the sample area. See Rule of 100 feet for estimating the extent of level 1 block/transverse cracking.



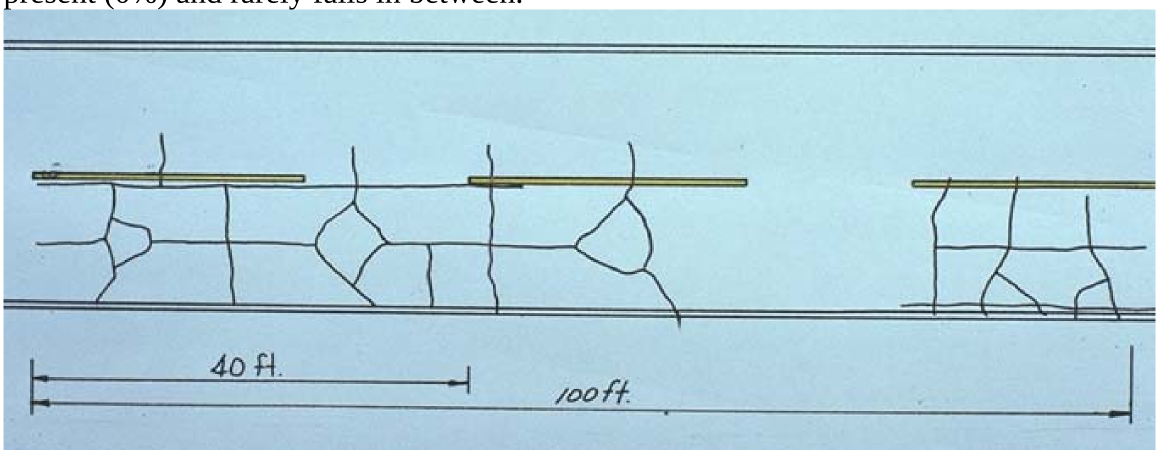
Examples of Level 1 Block/Transverse Cracking



Severity Level 2 Block/Transverse Cracking

At this severity level, the cracking has developed definite block patterns. Some of the longitudinal cracks can occur in the wheelpaths for short distances without being considered load cracking when associated with a block pattern. The transverse and longitudinal cracks are wider than in level 1, but do not necessarily require sealing. The block pattern will usually be uniform across the entire roadway.

In the following example, 80% of the sample area has level 2 block/transverse cracking. However, this example is for illustrative purposes only. It is either present (100%) or not present (0%) and rarely falls in between.



Following are sample images of level 2 block/transverse cracking.

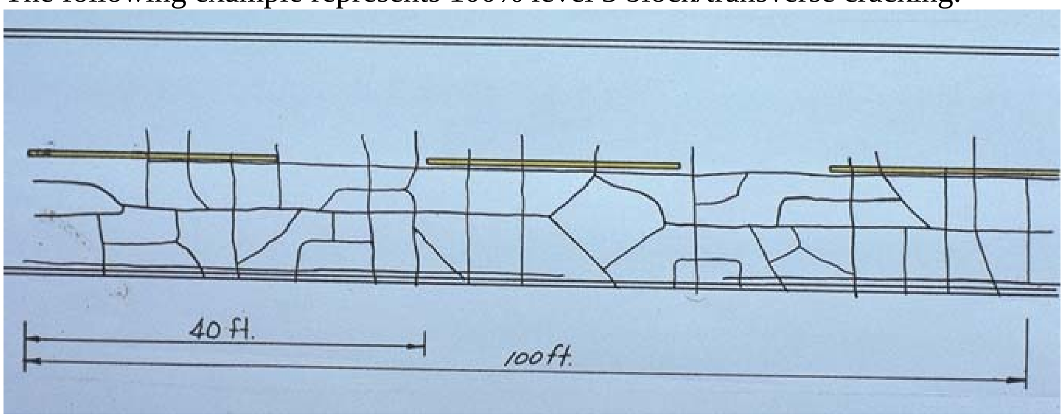
APPENDIX E

Examples of Level 2 Block/Transverse Cracking

Severity Level 3 Block/Transverse Cracking

At this severity level, the cracking has a definite block pattern as in level 2, but the blocks are smaller and the cracks are wider than in level 2. Level 3 block/transverse cracking is marked by cracks that are wide enough to require sealing. Block cracking that has a very large number of small blocks is also considered to be level 3 block/transverse cracking. Some of the longitudinal cracks in this pattern may meander into the wheelpaths for short distance, but are still considered block cracking because large distances of wheelpaths are not affected and this type cracking is not caused by loading. Some spalling of the cracking may be evident.

The following example represents 100% level 3 block/transverse cracking.



Here are some sample images of level 3 block/transversecracking.

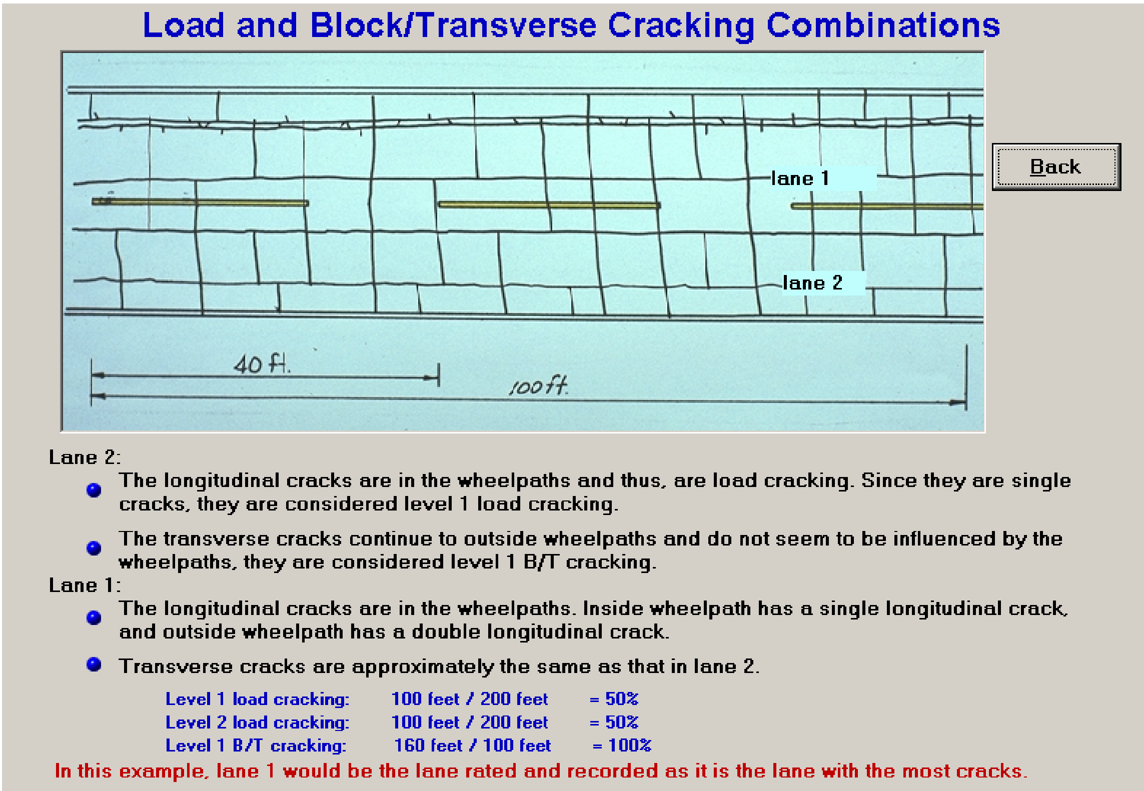
Level 3 block/transverse cracking – Notice width, subsidence and spalling of the cracks

Load Cracking and Block/Transverse Cracking Combination

The combination of load cracking and block/transverse cracking occurs to some extent on practically all roads, especially after a few years of traffic and weathering, However, in most cases, the combination will consist of load cracking (levels 1-4) and block/transverse cracking (level 1 only).

In a **few** cases, such as an older, heavily weathered and oxidized road which is subjected to a sudden increase in traffic, especially trucks, such roads can have load cracking (levels 1-4) and block/transverse cracking (levels 2-3) occurring as a combination.

Such a combination is the **exception** to the rule and the rater should double-check to insure that such a combination does, in fact, exist. The more likely result is an error by the rater in "double-counting" of the longitudinal cracks as both a load crack and as part of the “block” pattern, thus, calling it either level 2 or 3 block/transverse cracking. **A longitudinal crack in the wheelpath is a load crack and is to be rated as such.** This crack has then been accounted for and **cannot** be used again to form the “block” pattern.

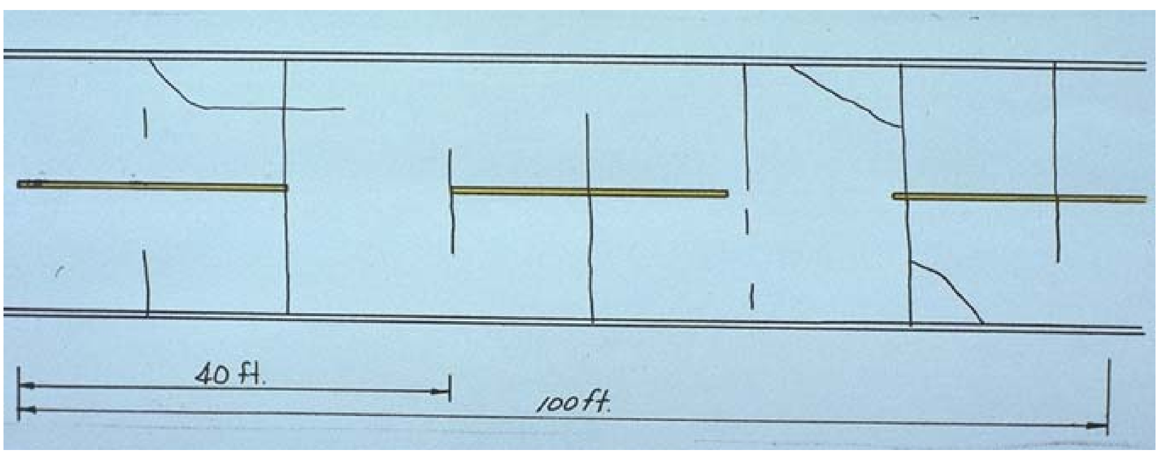


Reflection Cracking

This type cracking is caused by the “reflection” of joints and cracks through an asphaltic concrete overlay from the **underlying PCC concrete pavement**. These reflection cracks begin as tight cracks and progress to very wide cracks with spalling. The transverse cracks will be right angles across the roadway and in a repeatable pattern down the roadway (i.e., every 30 feet, 40 feet, etc.). The longitudinal cracks, if present, will normally be fairly straight, continuous cracks in the travel lane near the pavement edge associated with underlying edge of narrower PCC concrete pavement which has been widened and overlaid with asphaltic concrete overlay. Longitudinal cracks that occur at the centerline, lane lines, and edge lines are not to be counted. Any other cracks will be cracks associated with failures in underlying PCC concrete pavement and such cracks will reflect the size and shape of such failures. Construction joints and widening joints associated with the construction and/or widening of asphalt pavements are not to be counted as reflection cracking. Remember, reflection cracking only occurs on roadways with an underlying PCC concrete pavement.

Severity Level 1 Reflection Cracking

Level 1 Reflection cracks are tight, single cracks that are usually transverse, but are longitudinal if the underlying concrete pavement is narrower than the asphaltic concrete overlay. Irregular cracking patterns can be reflected if the underlying slabs are broken. Level 1cracks may or may not go across the entire lane.



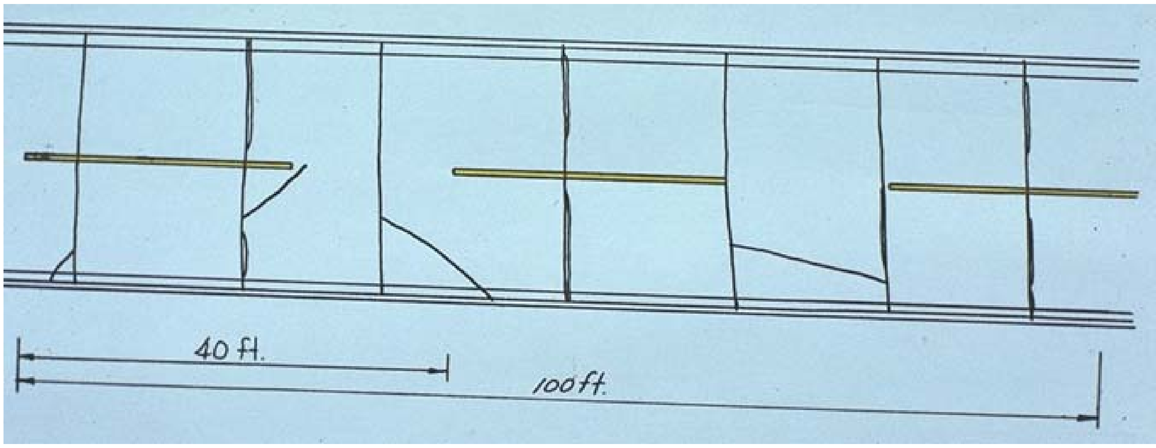
This illustration shows the top lane represents 9 total cracks and approximately 100 linear feet of reflection cracking.

APPENDIX E

Here are some sample images of level 1 reflection cracking

Severity Level 2 Reflection Cracking

This level of reflection cracking has progresses so that all underlying joints and cracks have reflected through the surface layer. The cracks are substantially wider than the level 1 cracks and may require sealing. There may be some "double" cracks over the underlying concrete pavement joints. "Double" cracks are not to be counted as two cracks, but one reflection crack. A longitudinal crack in the travel lane near the edge of pavement that is a result of widening of the underlying concrete pavement with asphalt will be counted as a reflection crack. If the underlying pavement is not concrete, do not count. A widening crack is not necessarily a reflection crack.



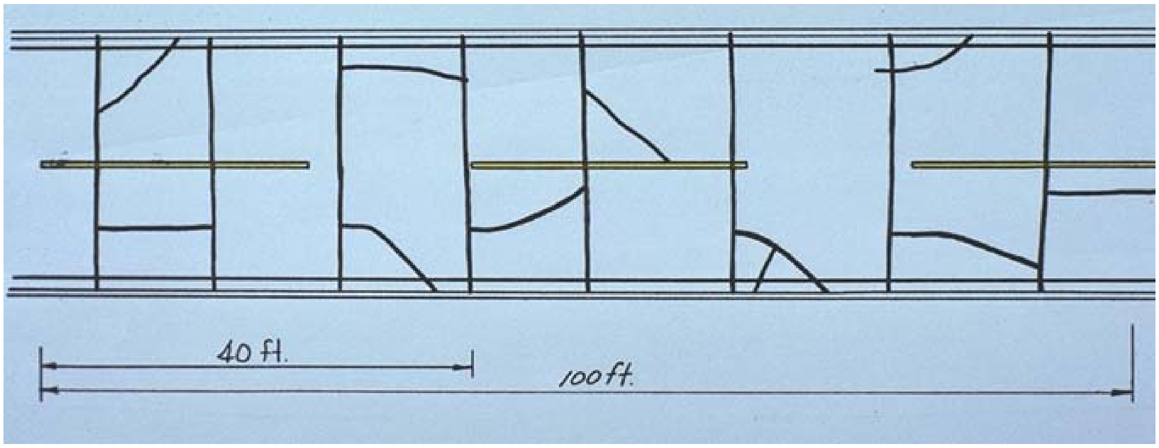
This illustration the bottom lane represents 12 total cracks and approximately 230 linear feet of reflection cracking.

Here are some sample images of level 2 reflection cracking.

Level 2 Reflection.

Severity Level 3 Reflection Cracking

This level of reflection cracking will have the same pattern as level 2 reflection cracking (all underlying joints and cracks have reflected through), but the cracks will be very wide. The cracks will be marked by spalling and/or subsidence. It should be obvious that some corrective work should be performed to these cracks before counting them as level 3.



This illustration shows the bottom lane represents 16 total cracks with approximately 280 linear feet of reflection cracking.

Here are some sample images of level 3 reflection cracking.

Raveling

Description:

This condition is the progressive disintegration of the pavement surface. It is caused by traffic action on a weak surface. Aggregate particles become dislodged from the binder and this loss of material can progress through the entire layer. Raveling ranges in severity from the loss of a substantial number of surface stones to the loss of a substantial portion of the asphalt surface layer. For purposes of rating, a slurry seal that has "peeled off" is considered level 3 Raveling.

How to measure:

The percent of the length of the rated segment (mile or partial mile) that contains raveling is to be recorded along with the predominant severity level. For example, if you observed 3000 feet of raveling in a 1 mile rating segment with 1000 feet rated as level 1 severity and 2000 feet rated as level 2 severity, you would record as 60% level 2 raveling as 3000/5000 = 60% and level 2 is more predominant (2000 ft. > 1000 ft.).

Level 1 – loss of substantial number of stones

Level 2 – loss of most surface

Level 3 – loss of substantial portion of surface layer (>1/2 depth)

Edge Distress

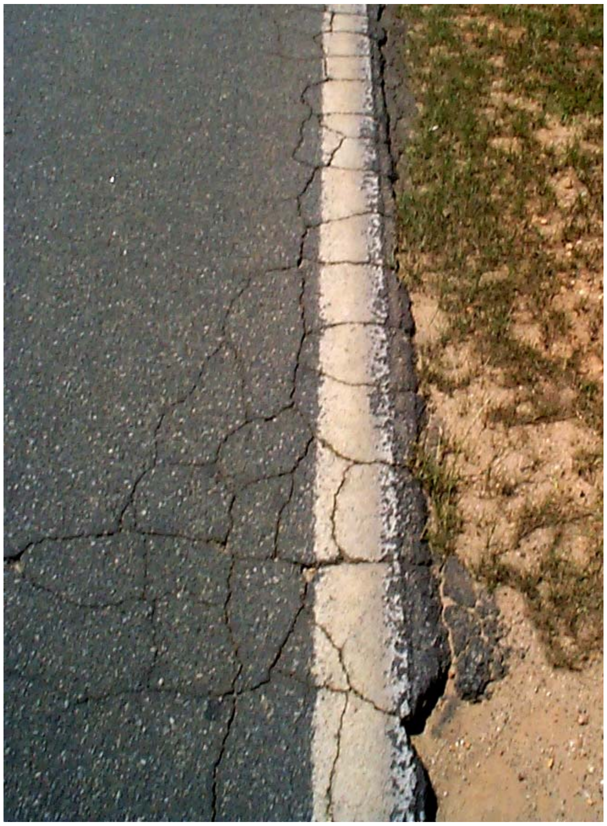
Definition:

Edge distress is cracking and pavement edge break-off within 1 to 2 foot of the pavement edge and not associated with the wheelpath area. The cracking can be in the form of longitudinal or transverse cracks or in many instances alligator type cracking. It may sometimes be difficult to distinguish between alligator cracking in the wheelpath and along the edge of the pavement especially on narrow width pavements. **It must be called load cracking when it occurs in the wheelpath. It cannot be called both load cracking and edge distress.**

How to Measure:

The percent of the rated segment (mile or partial mile) length containing edge distress is recorded on the survey form along with the predominant severity level in the rater's judgment. For example, if you observed edge distress on these curves (curve 1 --700' level 1, curve 2 –1000' level 2, and curve 3 –300' level 3) within a mile rating segment, you would record as 40% level 2 edge distress as 700 + 1000 + 300 = 2000/5280 = 40% and level 2 was more predominant (1000' > 700' > 300').

Level 1 – tight, hairline cracks



Level 2 – crack widths greater than ¼ inch, double cracking, tight “alligator” cracking

Level 3 – severe “alligator” cracking at edge, popouts, edge break off

Bleeding/Flushing

Definition:

Bleeding or flushing is the presence of bituminous material on the surface creating a shiny appearing. Bleeding or flushing is created by excess asphalt cement and/or low air void content.

How to measure:

The percent of the length of the wheelpaths that has bleeding or flushing in the rated segment (mile or partial mile) is noted. Each wheelpath is a maximum of 50 percent of the rated segment (mile or partial mile). For example, if you observed that one wheelpath had 3000 feet level 1 bleeding and other wheelpath had 1000 feet level 2 bleeding in a rated segment of 1 mile, you would record as 40% level 1 bleeding/flushing since 3000 ft. = 30% + 1000 ft. = 10% or 40% total both wheelpaths and level 1 is more predominant (3000' > 1000').

Level 1 – free bitumen is noticeable on the surface along with the aggregate in the mix

Level 2 – surface is black with very little aggregate noticeable

Corrugation/Pushing

Definition:

A series of ridges and valleys in the surface which cause a rippling or washboarding effect caused by unstable asphalt on asphaltic concrete or non-uniform application of aggregate on surface treatment.

How to Measure:

The extent will be recorded as the percentage of the rated segment (mile or partial mile) length that has corrugations. For example, if you observed in a mile rating segment that three interchanges (No. 1 –700 feet level 1, No. 2 –800 feet level 2, and No. 3 –500 feet level 3) had corrugations/pushing, you would record as 40% and level 2 corrugation/pushing, as 700 + 800 + 500 = 40% and level 2 is more predominant (800’ > 700’ > 500’).

Level 1 – corrugations are visible and can definitely be felt in the steering wheel while driving

Level 2 – corrugations/pushing have significant effect on riding comfort, some reduction of speed may be necessary

Level 3 – noticeable discomfort, excessive vibration, reduction of speed necessary

Loss of Section

Definition:

A deviation of the pavement surface from its original typical design cross section other than those described for corrugations, pushing or shoving. Generally, loss of pavement section results from settlement, slope failure, or heavy loads on a deficient pavement system. This loss of section usually occurs in the outside half of the lane. Loss of section takes the form of dips, bumps, and undulations, all of which cause pitch and role in a moving vehicle.

How to Measure:

The percentage of the length of rated segment (mile or portion of mile) that has loss of pavement section. The three severity levels are as follows:

Level 1 (Slight): Noticeable swaying of vehicle, but no effect on vehicle control.
Level 2 (Moderate): Heavy swaying of vehicle. Fair control of vehicle, driver has to anticipate dips ahead.
Level 3 (Severe): Speed of vehicle must be greatly reduced for driver to maintain control.

The predominant severity, in the rater's judgment, is also recorded on the rating form under severity. For example, in a rating segment of one mile, you observe 3000 feet level 1 and 2000 feet level 2, you would record as 100% level 1 loss of pavement section as 3000 + 2000 = 100% and level 1 is more predominant (3000' > 2000').

Level 1 (slight) – Noticeable swaying of vehicle, but no effect on vehicle control

Level 2 (Moderate) – Heavy swaying of vehicle. Fair control of vehicle, driver has to anticipate dips ahead

Level 3 (Severe) – Speed of vehicle must be greatly reduced for driver to maintain control

Patches, Potholes, and Local Base Failures

Definition

Patches are repaired sections of the asphalt pavement due to localized pavement and/or base failures. Also spot leveling is included in this category as well.

Potholes are sections of the asphalt pavement that have failed and formed a hole in the pavement structure. These are caused by pavement and/or base failures.

Base Failures are sections of roadway where the water has entered the base material and is rutting and shoving .

How to measure

The total number of spot overlays, patches, potholes and local base failures must be counted for the entire rated segment (normally a mile) and this number recorded on the COPACES survey form. Utility patches are NOT to be counted UNLESS the utility patch has failed and is thus affecting the structural condition of the pavement.

Rating Survey

Introduction

Ratings are done for each mile (or partial mile) by selecting a sample section for cracking distresses representative of the pavement condition for that rating segment. The defects noted for each rating segment within a project are then averaged to obtain the representative pavement condition for that project. A project rating is then determined from deduct values which have been established for each defect and severity level.

Conducting the Survey

The rating system has been devised so that the pavement condition of all the state routes can be assessed objectively by one rater within his assigned area on a project basis. It is suggested that the rater rates all the state routes within one given area at a time to reduce driving time between routes and to keep track of what areas have been surveyed. It would be helpful to mark on a map these routes that have been completed.

The rater must start the route at the beginning or ending point of the selected project limits and continue the rating process until the other end is reached. Do not survey any route by starting in the middle and do not branch off to another route prior to completing the route that is being rated. For roads with more than one road number, the lowest number description should be used. A programmable Distance Measuring instrument connected to the survey vehicle should be used when conducting the survey to accurately identify the location of each survey site.

Project Limit Selection

Pavement condition ratings will be obtained on all state routes. The roads will be divided into projects for analysis of the data.

A project is a length of roadway with a common pavement section, similar structural conditions, and logical beginning and ending points. Project limits should not be mileposts, businesses or other points not readily located on the map. The rater must choose the project limits when conducting the inventory ratings within the guidelines described in this section.

The following must be used as break points for project limits:

1. Major changes in pavement condition for more than two consecutive miles.
2. Change in pavement type (not including spot overlays).
3. Common sections containing more than one state route – rate the section as lowest numbered state route.
4. Divided highways. **Divided highway projects will have two files per segment of roadway with the only difference in the files being that the mileposts will be in the positive direction for one and in the negative direction for the other.**

The following break points can also be considered for project limits but are not required:

1. Intersecting State Routes
2. County Lines – If the pavement type does not change at the county line, consideration should be given to extending the project limits into the adjoining county.
3. City Limits
4. Changes in the number of lanes
5. Curb and gutter sections through a city or town
6. Project limits established from previous resurfacing projects
7. Original construction project limits.

Other local factors may be known to the rater which can be helpful in establishing a project limit. Once the project limits have been established during the initial rating survey, these same limits should be used during all follow-up ratings as long as structural condition remains similar within the project limits. If structural conditions changes, subdivide into 2 or more projects, as appropriate. As a “Rule of Thumb”, whenever the project limit selected is longer than 10 miles, double-check to be sure that the pavement conditions are basically similar within the limits selected. If not, separate into two or more projects.

Selection of Rating Segments

A project will normally be divided into one mile segments for rating purposes. Exceptions to this are the beginning and ending segments of a project which can be less than one mile or when drastic changes in pavement conditions occur within the mile and shorter rating segments (usually ½ mile) are used to get a more representative rating of pavement conditions, especially cracking distress. The project limits within a city also will generally be shorter in length because of changes in pavement type, number of lanes, etc. As pavement conditions normally vary greatly within short distances, the rating segment likewise will be reduced to ½ mile or less to insure getting a representative rating of pavement condition.

Selection Sample Location for Cracking Distress

In rating cracking distress (load cracking, block/transverse cracking and reflection cracking) only a 100 foot sample out of each rating segment (mile or partial mile) will be rated so it is very important that the 100 foot section represent the majority of the cracking distress found in the rating segment (normally a mile). The 100 foot section will be chosen by the rater and can be located anywhere within the rating segment. The rater should drive slowly and make two or three stops within the first half of the rating segment and look at the pavement from the car to determine what types of cracking distress and level of severity is generally present. The 100 foot sample section should be selected only after the rater is confident that he has a “feel” for the pavement condition and can select his 100 foot sample section that is representative of the cracking distress within the segment to be rated. On projects where conditions are uniform the pavement condition may be obvious after the first stop and the sample section could be chosen early in the rating segment. On projects with variable conditions, the sample section should normally be located at the half way point or beyond in the rating segment to be sure it is "representative" of the cracking distress. If pavement conditions change drastically within the segment being rated (normally a mile), the rating segment should be broken into two or more segments and 100 foot sample locations chosen to represent the smaller segments. For instance, two 100 foot sample sections could be chosen, each representing ½ mile.

For projects with cracking distress that vary widely within each rating segments, the 100 foot sample should represent the average conditions within the segment rather than the best or worst general conditions. For example, if in a given mile there is a substantial amount of cut areas, and the fill areas are the worse general condition, the 100 foot sample should be chosen to represent the entire mile, not in the best or worst area.

The rater should not locate the 100 foot samples over culverts, bridge approaches or locations that are obviously localized problems. Localized problems should be handled in "remarks".

The purpose of the survey is to obtain a representative rating of the project pavement condition, especially cracking distress.

Rating the Sample Section for Cracking Distress

The Computerized Pavement Condition Evaluation System program (COPACES) is the computer program that GDOT uses to administer PACES. All project file and survey information is entered for the project while the rater is conducting the survey via a laptop computer.

Once the 100 foot sample section has been selected to represent the cracking distress, its location will be recorded in COPACES program under the field "sample location". This field locates the sample section to the nearest one-tenth mile.

The rater must walk the 100 foot section (three centerline stripes plus 10 feet is approximately 100 feet) and rate the lane in the worst condition on two lane and multilane undivided roads. On divided highways each travel direction must be rated separately, although only the lane in the worst condition in each travel direction is rated.

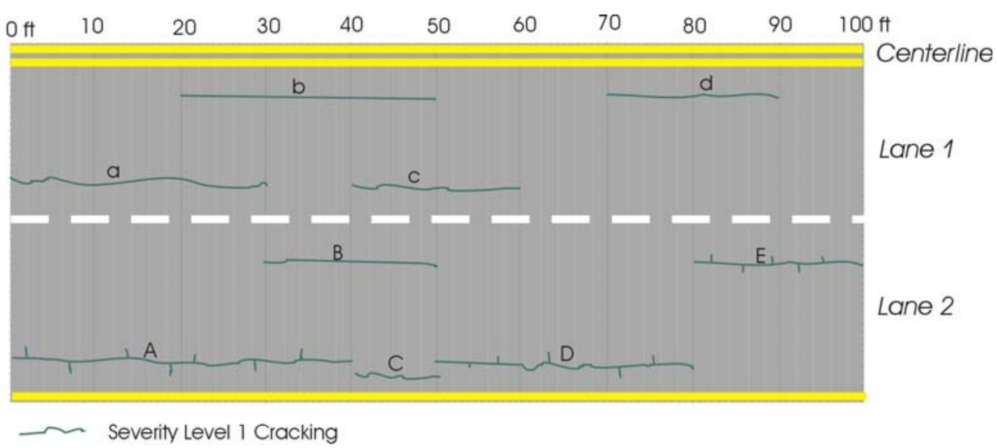
It is generally best to walk the 100 feet in one direction and determine which lane is in the worst cracking distress condition and rate this lane when walking back towards the vehicle. The rater must be aware that certain conditions such as time of the day, sunlight, and wetness can affect his ability to seen certain cracking distress conditions. The amount (to the nearest 5%) and severity of the cracking distress is estimated to the rater's best judgment in accordance with the procedures contained in Chapter 1 of this manual and immediately recorded in COPACES.

Selecting and Rating the Remaining Types of pavement Distress
As cracking distress is the most critical type of pavement distress, the length of the rating segment (mile or partial mile) will be determined when rating cracking distress as described earlier.

The remaining types of pavement distresses (Rut Depth, Raveling, Edge Distress, Bleeding/Flushing, Corrugation/Pushing, and Loss of Pavement Section) are to be closely observed and an estimate (to the nearest 5%) made of the extent and the predominant severity of the distress within the rating segment.

On two-lane and multi-lane undivided highways, the rater should determine which lane is in the worst general shape (from the standpoint of Raveling, Bleeding/Flushing, Corrugation/Pushing, Edge Distress, and Loss of Pavement Section) and base his estimate of the extent and severity of such pavement distress on what is observed in the lane selected. Likewise, on divided highways, only the lane in the worst condition in a given direction is to be rated, but rate both directions separately when rating divided highways (i.e., a separate report is to be prepared for each direction of a divided highway).

An exception to rating only the worst lane is to be made when rating patches, potholes, or local base failures as the total number of such distress for ALL LANES within the rating segment is to be recorded.



Lane 1 Survey
a : 30 ft
b : 30 ft
c : 20 ft
d : 20 ft

Total cracking in two wheelpaths: 100 ft
Percentage of Level 1 Cracking : 100 / 200 = 50 %

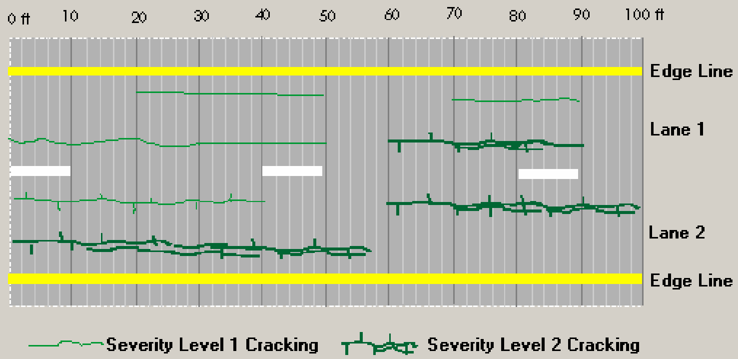
Lane 2 Survey
A : 40 ft
B : 20 ft
C : 10 ft
D : 30 ft
E : 20 ft

Total cracking in two wheelpaths: 120 ft
Percentage of Level 1 Cracking : 100 / 200 = 60 %

The higher percentage lane, 60 % of severity level 1 cracking in lane 2,
is used to represent cracking in this section.

Load Cracking - Combinations

The following figure illustrates the presence of level 1 and level 2 load cracking in lane 1 and lane 2.



Determine % Distresses:

In Lane 1

Level 1: (30 + 20 + 50) / 200 = 50%
Level 2: 30 / 200 = 15%

In Lane 2

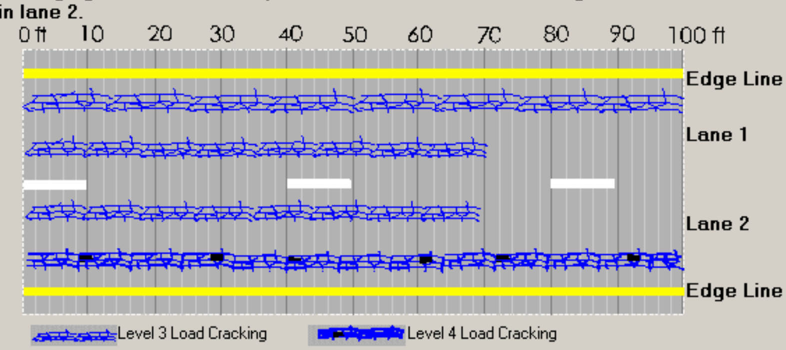
Level 1: 40 / 200 = 20%
Level 2: (40 + 60) / 200 = 50%

In this example, lane 2 would be the lane rated.

View Images | How to Determine % Distress | Exit | Back

Load Cracking - Combinations

The following figure illustrates the presence of level 3 load cracking in lane 1 and level 3 and level 4 load cracking in lane 2.



Determine % Distresses:

In Lane 1

Level 3: (70 + 100) / 200 = 85%
Level 4: 0 / 200 = 0%

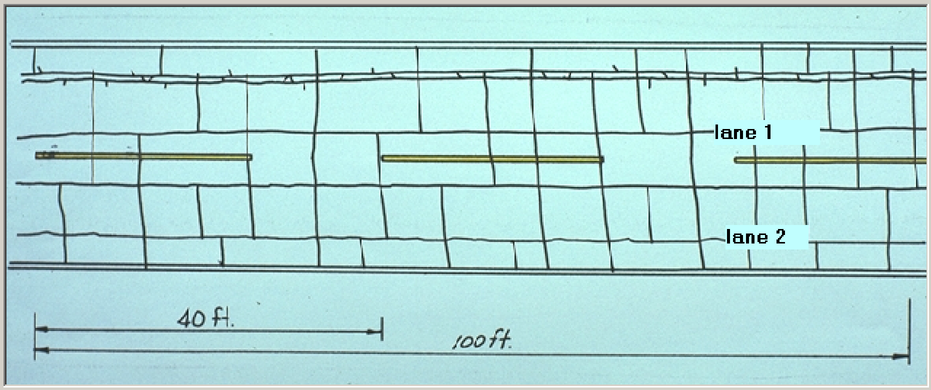
In Lane 2

Level 3: 70 / 200 = 35%
Level 4: 100 / 200 = 50%

In this example, lane 2 would be the lane rated.

View Images | How to Determine % Distress | Exit | Back

Load and Block/Transverse Cracking Combinations



Back

Lane 2:

- The longitudinal cracks are in the wheelpaths and thus, are load cracking. Since they are single cracks, they are considered level 1 load cracking.
- The transverse cracks continue to outside wheelpaths and do not seem to be influenced by the wheelpaths, they are considered level 1 B/T cracking.

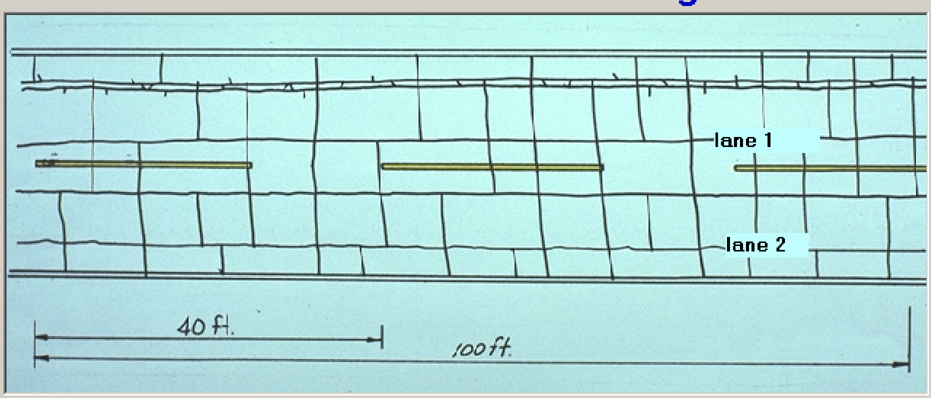
Lane 1:

- The longitudinal cracks are in the wheelpaths. Inside wheelpath has a single longitudinal crack, and outside wheelpath has a double longitudinal crack.
- Transverse cracks are approximately the same as that in lane 2.

Level 1 load cracking:	100 feet / 200 feet	= 50%
Level 2 load cracking:	100 feet / 200 feet	= 50%
Level 1 B/T cracking:	160 feet / 100 feet	= 100%

In this example, lane 1 would be the lane rated and recorded as it is the lane with the most cracks.

Load and Block/Transverse Cracking Combinations



Back

Lane 2:

- The longitudinal cracks are in the wheelpaths and thus, are load cracking. Since they are single cracks, they are considered level 1 load cracking.
- The transverse cracks continue to outside wheelpaths and do not seem to be influenced by the wheelpaths, they are considered level 1 B/T cracking.

Lane 1:

- The longitudinal cracks are in the wheelpaths. Inside wheelpath has a single longitudinal crack, and outside wheelpath has a double longitudinal crack.
- Transverse cracks are approximately the same as that in lane 2.

Level 1 load cracking:	100 feet / 200 feet	= 50%
Level 2 load cracking:	100 feet / 200 feet	= 50%
Level 1 B/T cracking:	160 feet / 100 feet	= 100%

In this example, lane 1 would be the lane rated and recorded as it is the lane with the most cracks.

Chapter III File Management

PACES surveys are performed using the most current version of the Computerized Pavement Condition Evaluation System (COPACES). File Management of COPACES files is critical for rater to manage his COPACES program successfully. Special care should taken by the rater to ensure that computer files are stored properly and that none are missing, obsolete, or duplicated.

In order to practice proper file management procedures, it is essential to have a basic understanding of how COPACES operates. When a file is created in COPACES, that file is created as a Microsoft ACCESS database file. The file name is arranged in a special format that needs to be understood by the rater. By understanding this format, the rater should be able to easily identify files and their location.

In COPACES application, the database file name is given according to the following predefined format:

A_NNNN_XX_OO_FF1_BBBBB_EEEEE_FF2_BBBBB_EEEEE_FF3_BBBBB_EEEEE_MM_DD_YYYY_TT_MI_SS.mdb

A – District No.
NNNN -- Route No.
XX – Route Suffix
OO – Office
FF1 – County # of County 1 (the first county)
BBBBB -- Mile from (from which the corresponding project is conducted.)
EEEEE -- Mile to (by which the project is to end)
FF2 – County # of county 2 (if available)

FF3 – County # of the county 3 (if available)
MM – Month
DD – Day
YYYY - Year
TT – Hour
MI -- Minute
SS – Second

Note:
File extension (.mdb) -- Microsoft access database file type
Time records (MM_DD_YYYY_TT_MI_SS) -- the time when the database file was first created.

For example, consider the following created database file:
1_0020_00_AO_241_00000_02456_000_00000_00000_000_00000_00000_11_01_1998_12_30_04.mdb

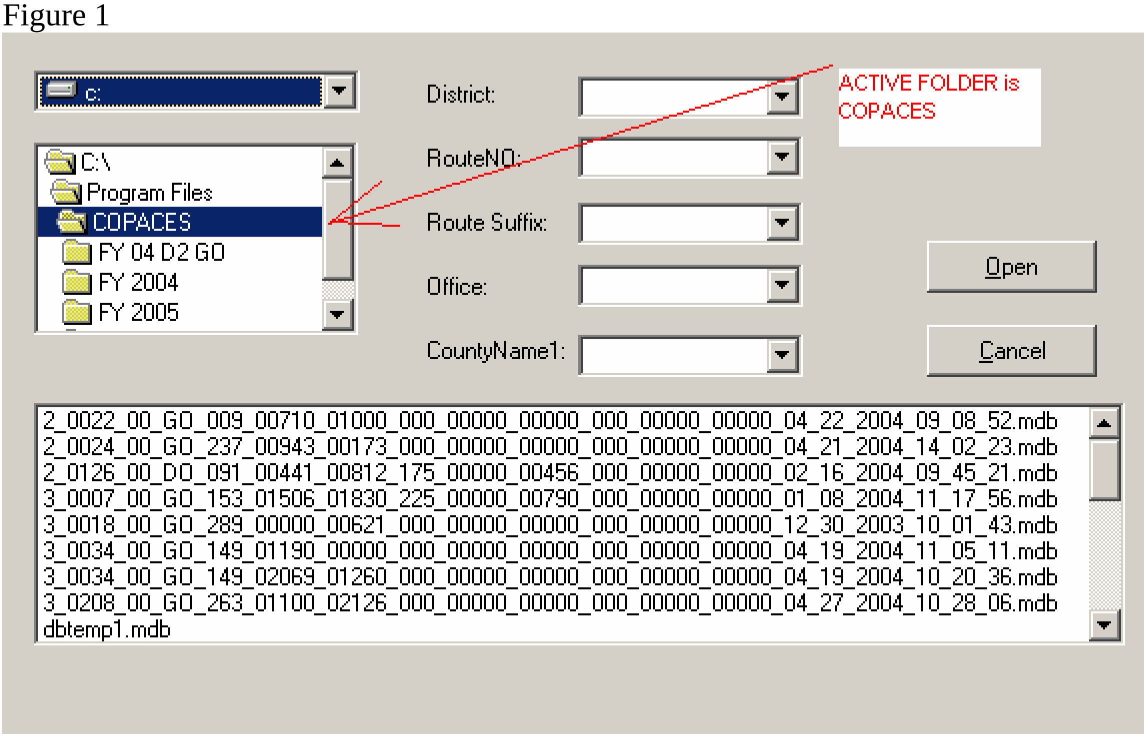
This database file was created at 12:30:04 pm on Nov. 01, 1998, corresponding to the project conducted by the area office (AO) on the State Route 20 (route No. is 0020, suffix is 00) in the county with County # 241(i.e. Rabun) in District No 1. The measured range is from 0.00 mile to 24.56 mile. Since the project was conducted only within the Rabun County (County # is 241), the county 2 and county 3 locations as well as the measured range are all set to 0.

File Management at the Area Office level.

When you are operating in the COPACES program, you are operating off of the hard drive on your computer at C:\Program Files\COPACES (see fig.1). Any new or updated file that you are working on will be saved by the program at this location.

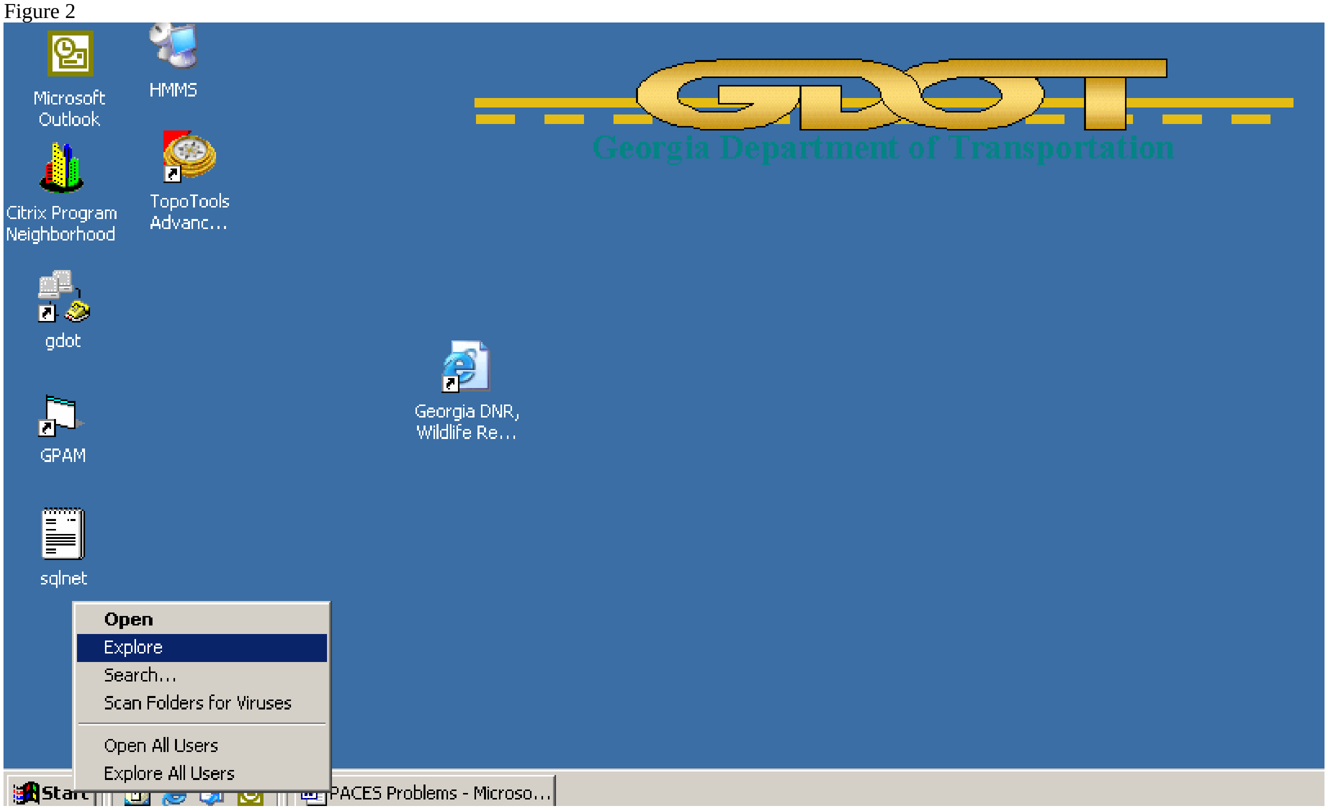
For example, you cannot directly save a file to the folder FY 2004 from the COPACES program. It is automatically saved in the COPACES folder.

Figure 1



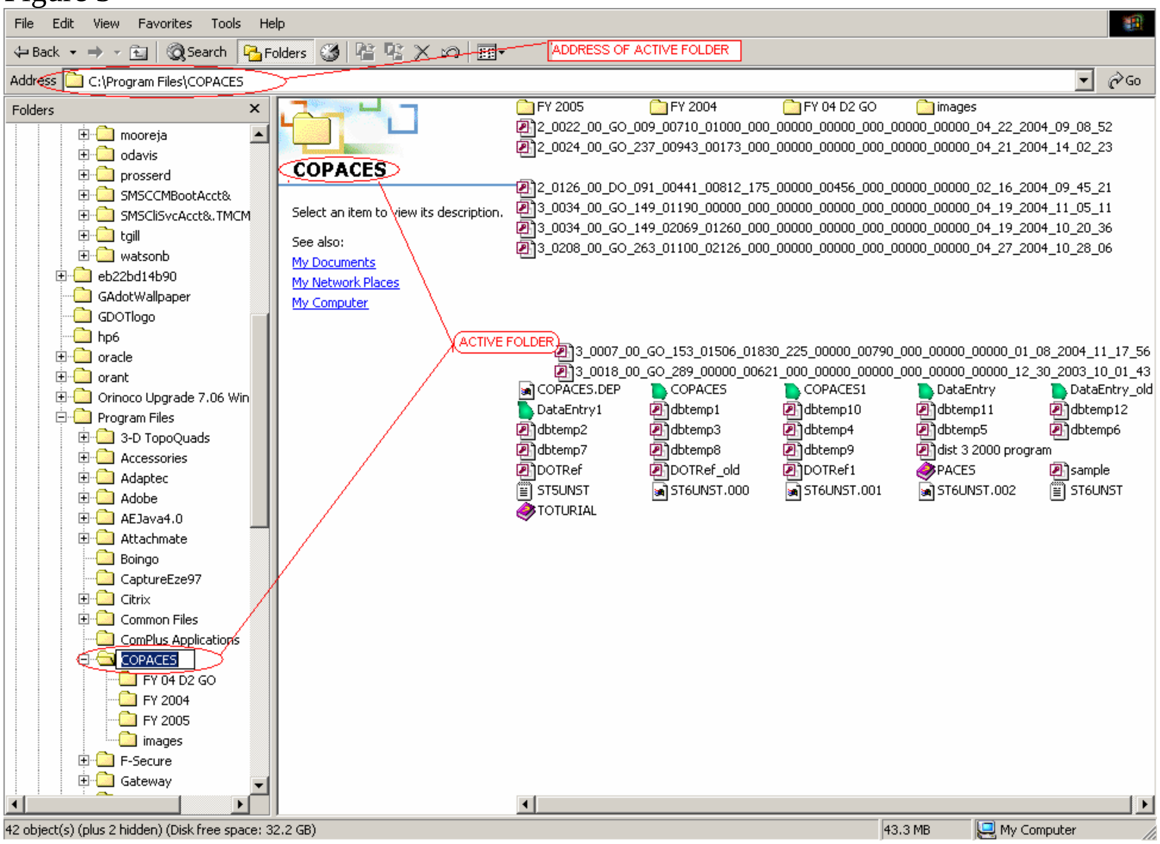
For this reason, a sub file should be created to store the completed files for the *current rating year* (C:\Program Files\ COPACES\FY 2004). **The goal is to have a folder for each rating fiscal year.** Once you have completed a route, you should move the file into this location. This will help you keep track of what you have completed and what you still have left to do. **It is important to remember that moving or deleting files, creating folders, etc. cannot be done while in the COPACES program itself**. These functions must be performed by right clicking on the START button and clicking on "Explore". For the purposes of this manual, this step is referred to as "going through Explore". See figure 2.

Figure 2



Once this is performed you will see a screen as shown in Figure 3 below. Notice that the COPACES folder is set up the same as in Figure 1. Also notice the address of the COPACES folder(C:\Program Files\COPACES).

Figure 3



Continuing with this example, the next rating season, you will create another sub file at C:\Program Files\COPACES\FY 2005. You should then **copy** all of your files from the 2004 folder into the COPACES folder (Do not copy to the FY 2005 folder). Using these files, you perform your COPACES ratings. Remember to update the information in the project file at the project location screen. Once each file is completed, you then move it to the FY 2005 folder as described previously. The COPACES folder also holds program files that run the program itself. Be careful not to move or delete these files.

After you have completed your ratings for the year, all of your files should be in the folder for that year. However, this does not mean duplicated and/or obsolete files. There should be only one file per segment of roadway except for divided highways. **Divided highway projects will have two files per segment of roadway with the only difference in the files being that the mileposts will be in the positive direction for one and in the negative direction for the other.** Extreme care must be taken to ensure that only valid files are in the FY 2005 folder. All other files must be deleted.

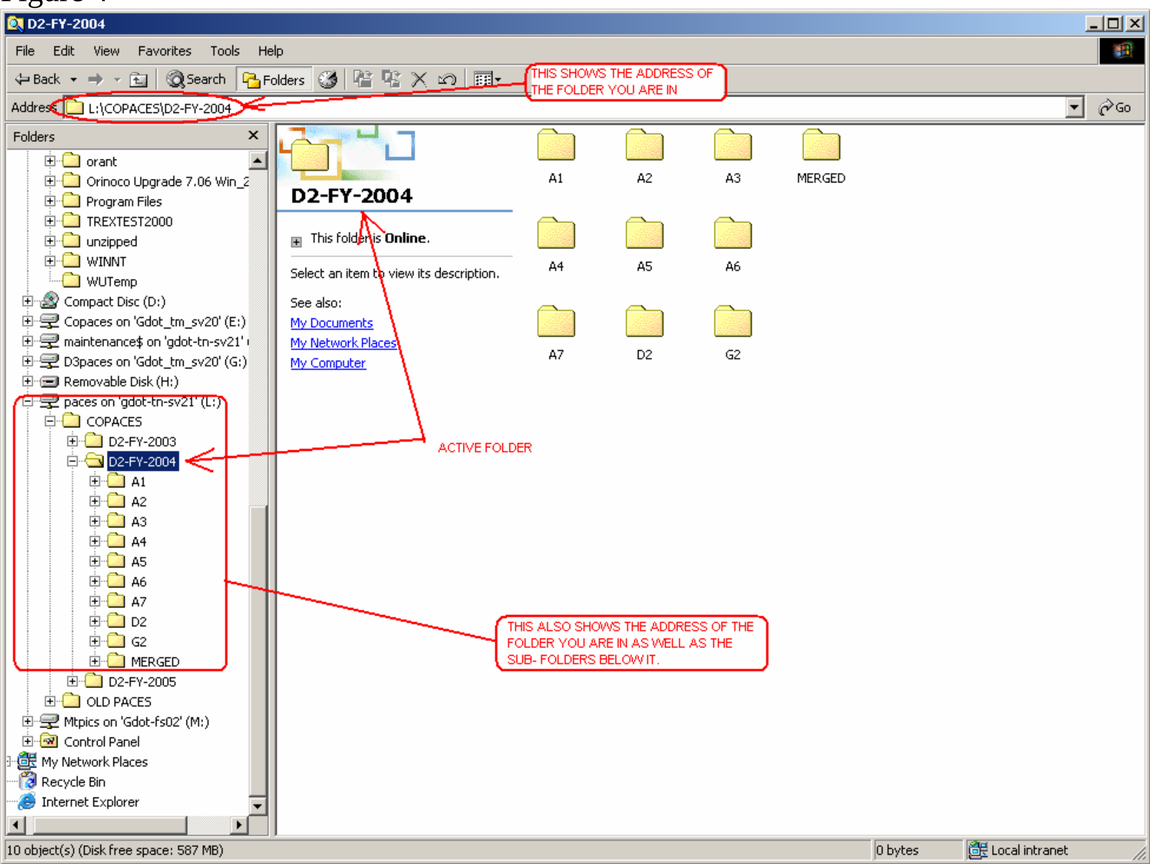
Once you have ensured that your files are correct, you are ready to export to the district server. Be sure that you only ***copy*** your files to this location. Remember, you still want to have the files on your computer for future reference.

Copying files to the District Office level is explained in the following section.

File Management at the District Office level.

File management for the District Office level is mandated by the State Maintenance office to be in the standardized format as seen in Figure 4(This example is from District 2). The purpose is to provide consistency within the districts so as to facilitate the retrieval and uploading of the data by the central office personnel.

Figure 4

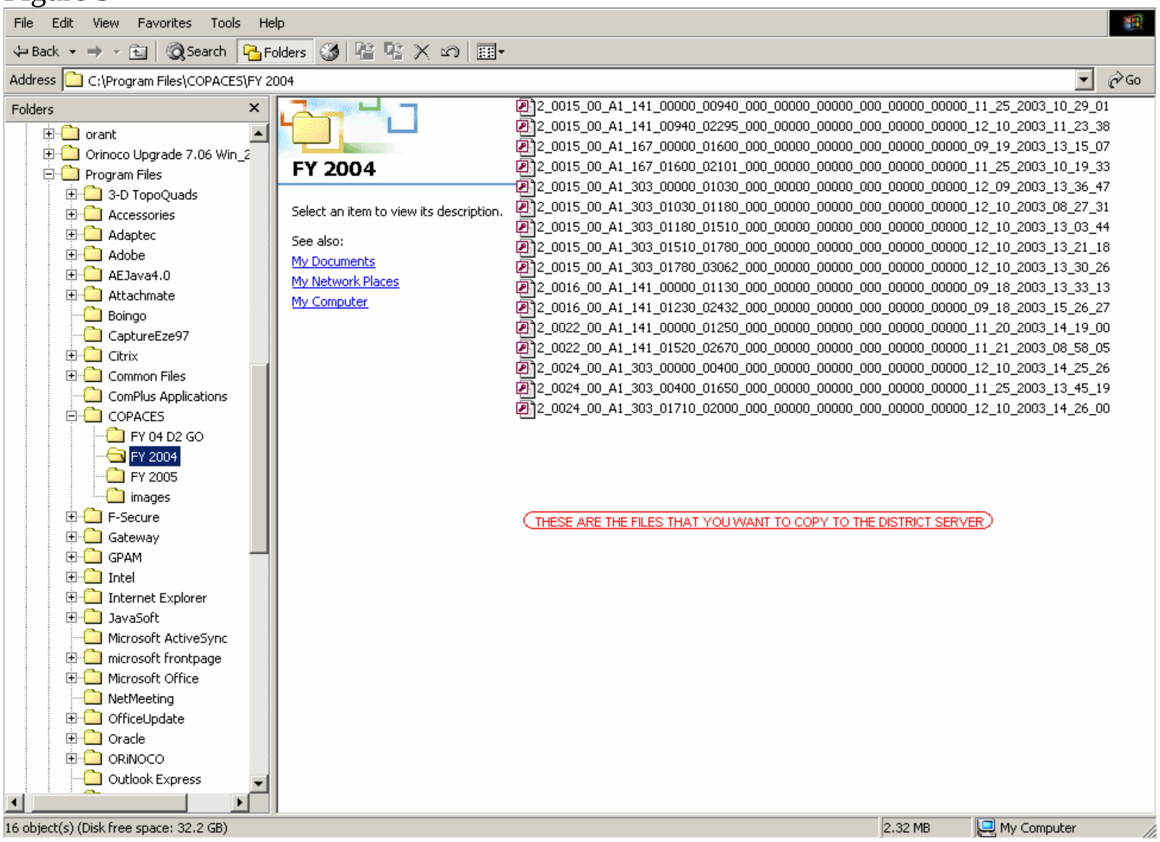


Once an area office has completed their ratings, they should upload the files to the appropriate area folder on the district server. The District office and the maintenance liaison should keep their ratings in the folders D# and G# respectively.

One way to copy or transfer files by the Area Office to the District Office is explained below.

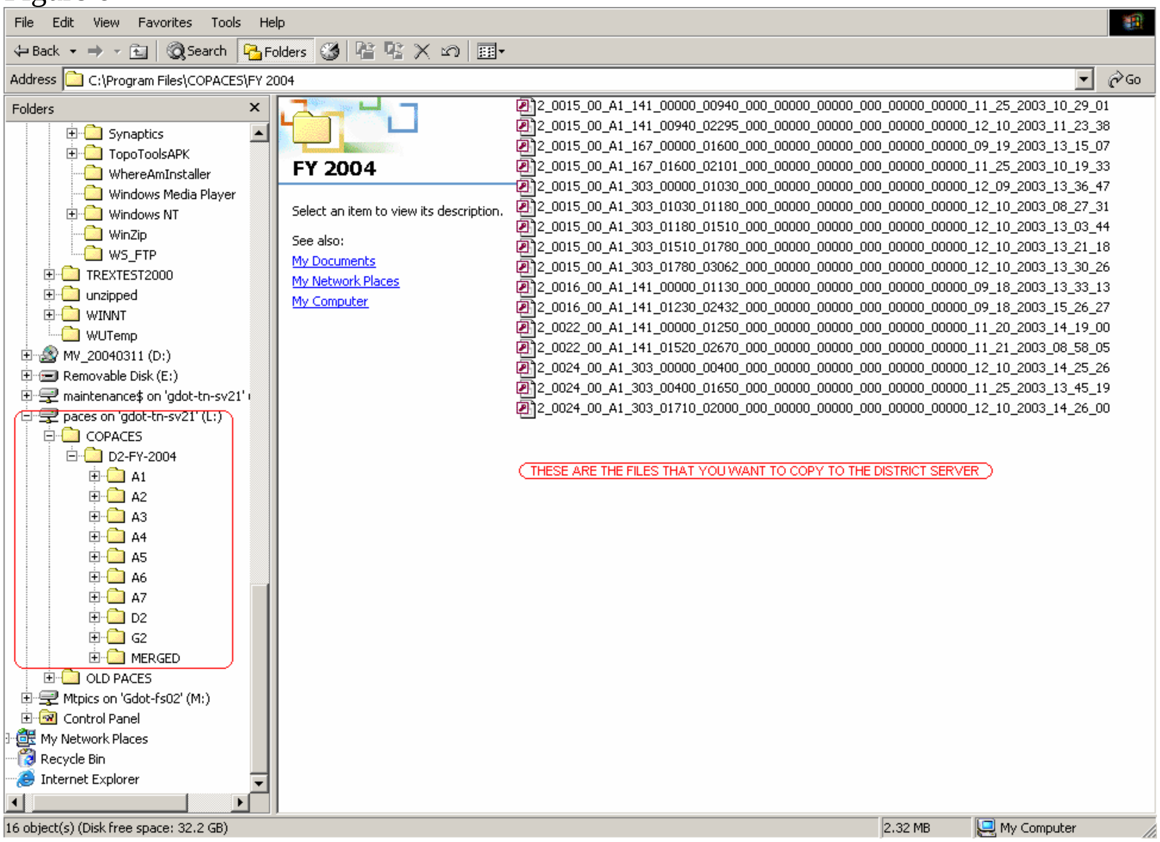
The first step is to go through Explore and open up the screen so that C:\Program Files\COPACES\FY 2004 is open on the right hand portion of the window as shown in figure 5.

Figure 5



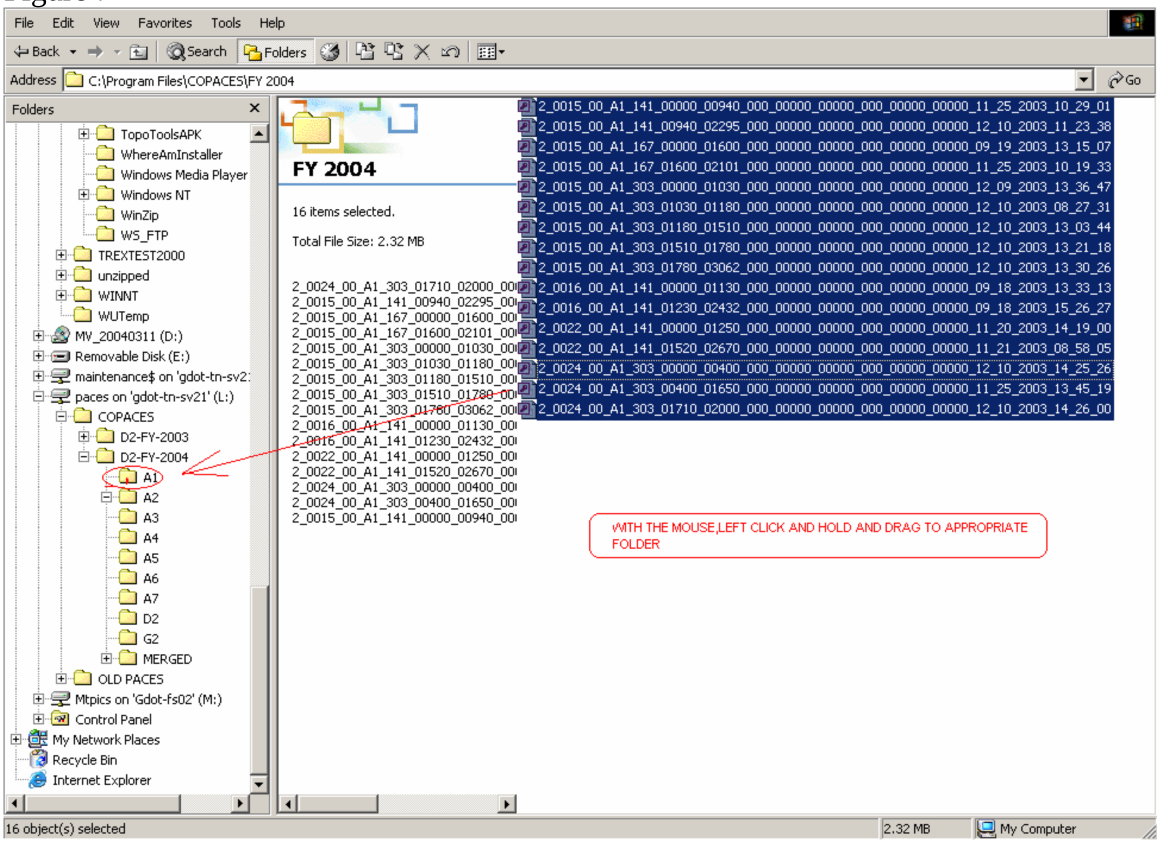
Then the folders that are on the District server should be opened up in the left portion of the window by clicking on the plus(+) in front of the folders until the left window looks like figure 6.

Figure 6



Once this is done, the files to be copied should be highlighted by holding the SHIFT key down and selecting the files with the mouse. Then, highlight the files by left clicking and holding the button down, and with the mouse drag the files to the appropriate folder in the left window. Release the button on the mouse and the files will be copied. See figure 7.

Figure 7



District Office File Management Responsibilities

Once all of the area office files have been uploaded to the district server site, it is the responsibility of the District Maintenance Engineer and his assistant to ensure that all routes have been surveyed, not duplicated and any erroneous files have been purged from the server location.

Copaces Data Quality Insurance program has been supplied by the State Maintenance Office, referred to as CoPaDQI, to accomplish this task. This program will help the district office detect duplicated route files and routes that were not surveyed.

Once the District Office is assured that the COPACES files are correct, the District Office should perform the Merge function in the COPACES program and locate the completed file in the Merged folder for the appropriate fiscal year on the District's server site.

Chapter IV Calculation of the Project Rating

A general understanding of the PACES calculation of the project ratings by the rater is essential for the rater to understanding the COPACES program. The COPACES program is based on the PACES system that was manually calculated by the rater.

The project rating obtained from PACES can vary from 0 to 100 points. One hundred points is assigned to a roadway with no visible surface distresses. Points are deducted from a possible 100 based on extent and severity of each surface distress. One hundred minus the total deduct points is the project rating.

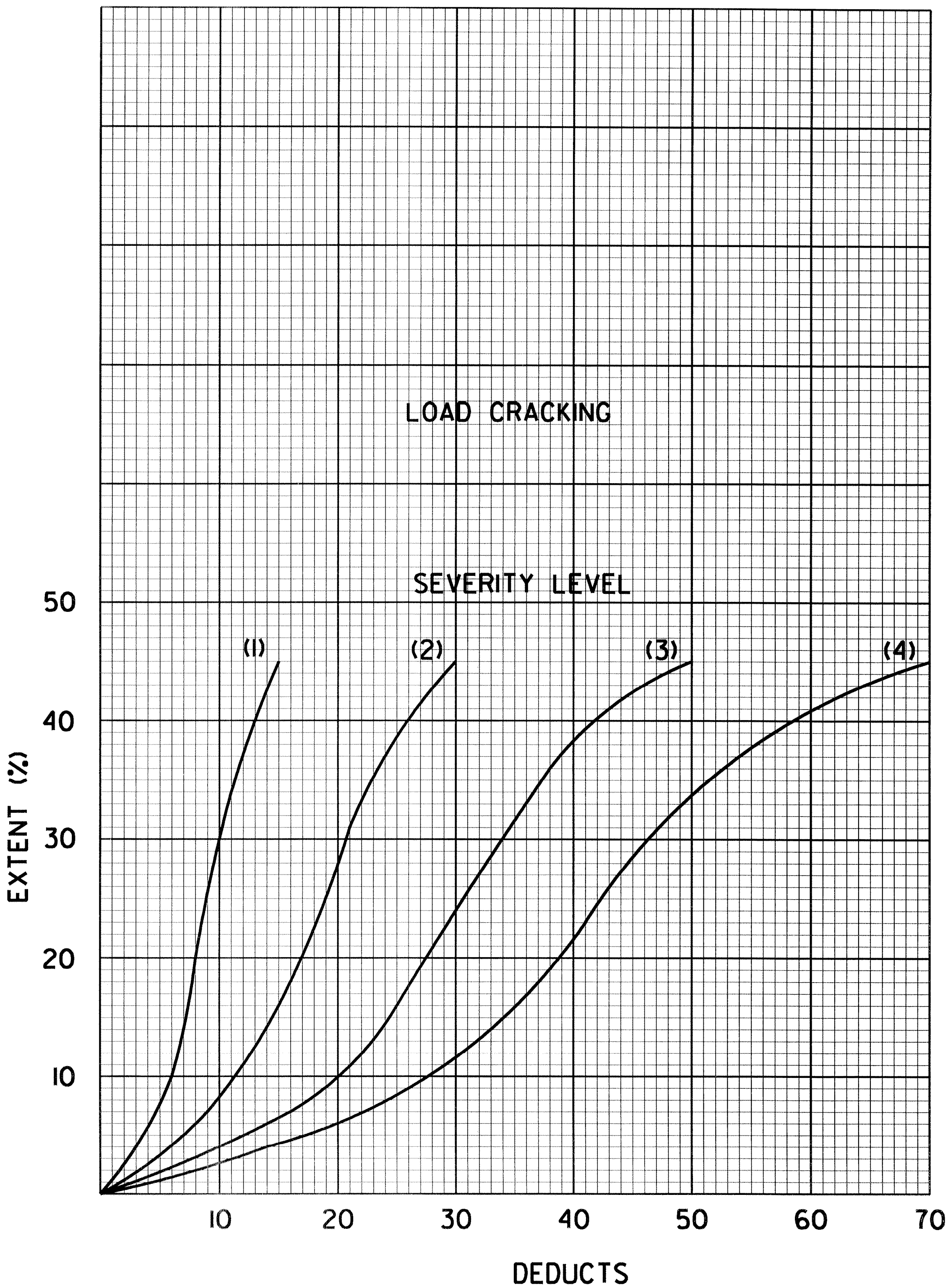
The deduct values are assigned based on average extent and predominant severity level for the entire project. **This fact points out the necessity of choosing projects properly. For example, a poor section of roadway, rated together as a project with a good section, will result in the poor section of the roadway not being adequately represented by the rating score.** Obviously, one or two miles cannot be separated from the middle of a project for special scoring. However, after close inspection of the Detailed Project Rating Sheet obtained from COPACES, it should be obvious if the project limits were chosen correctly or incorrectly. Consideration should be given to breaking the project into two projects if the conditions warrant.

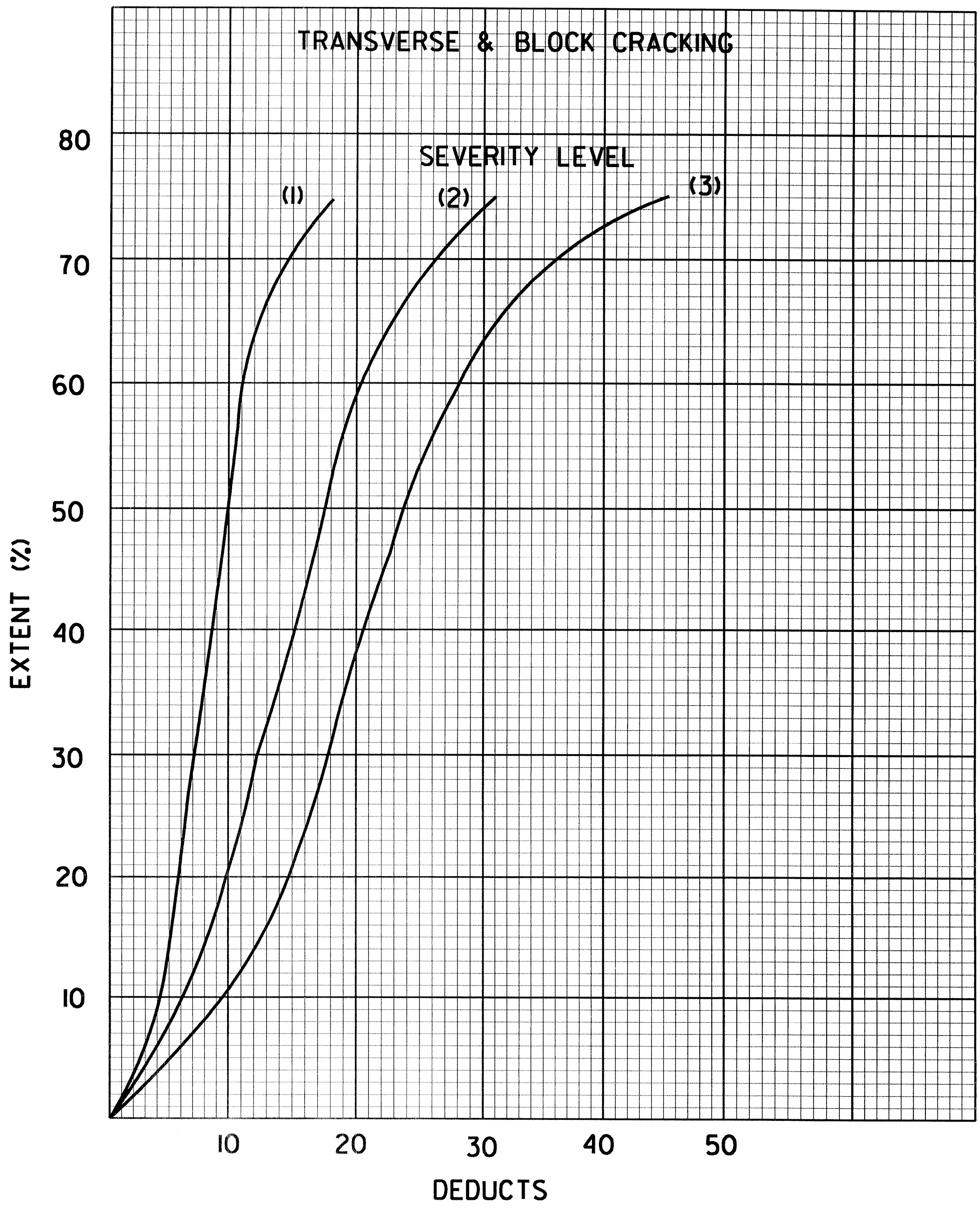
Determining Project Average for Each Distress

Simple numeric averages for each distress are used instead of prorating in this rating system. The averages are computed by totaling the values for each type of distress and dividing by the number of rating segments.

After the average values are computed for each distress for the project, deduct points are determined for each distress extent and severity. These deduct points are totaled and subtracted from 100 to determine the project rating.

The following charts, used when PACES was performed manually, are representative of the deduct point values used in COPACES.


LOAD CRACKING
SEVERITY LEVEL
(1)
(2)
(3)
(4)
EXTENT (%)
10
20
30
40
50
60
70
DEDUCTS


TRANSVERSE & BLOCK CRACKING
SEVERITY LEVEL
(1)
(2)
(3)
EXTENT (%)
80
70
60
50
40
30
20
10
10
20
30
40
50
DEDUCTS

Flexible Pavement Condition Survey Deduct values

Rutting Extent (inches)							
	0	1/8	1/4	3/8	1/2	5/8	3/4
Deducts	0	2	5	12	16	20	24

	Patches and Potholes Extent (# per mile)				
	1-2	3-6	7-10	11-15	>15
Deducts	2	5	10	17	25

		Corrugations/Pushing Extent (%)		
		1-10	11-25	>25
Severity	1	1	2	4
	2	2	4	7
	3	3	6	10

		Reflective Cracking (%)			
		5-15	16-30	31-45	>45
Severity	1	3	5	6	8
	2	6	8	11	14
	3	8	12	16	20

		Edge Cracking Extent (%)			
		5-25	26-50	51-75	>75
Severity	1	1	2	3	4
	2	2	4	6	7
	3	3	6	8	10

		Raveling Extent (%)					
		1-5	6-15	16-25	26-35	36-45	>45
Severity	1	2	5	6	8	10	13
	2	4	8	11	14	17	21
	3	6	12	16	20	25	30

		Loss of Pavement (%)			
		0-25	25-50	50-75	75-100
Severity	1	0	1	2	3
	2	2	4	6	8
	3	6	5	10	12

		Bleeding or Flushing Extent (%)		
		1-10	11-30	>30
Severity	1	2	5	8
	2	5	10	15

F Appendix F: CPACES and Other Distresses

F.1 Introduction

The jointed concrete pavements on the Interstate and non-Interstate system continue to deteriorate through normal weather conditions and to the large volume of traffic they carry, especially trucks. Many of Georgia's older concrete pavements were designed and built without dowels at the joints to assist with load transfer. This combined with the presence of free water under the slabs and some of the base types used have spelled trouble.

Water gets under the slabs through cracks and failed joints and with the passage of heavy trucks a pumping action begins. This erodes the base material creating a void under the slabs. The slabs crack, joints and cracks spall, joint faulting or step-offs occur and shoulders adjacent to the pavement will sag and crack. Maintenance is then required to replace the broken slabs, fill the voids with grout, patch the shoulders, reseal joints and repair any spalling at the joints. Rough pavement or excessive joint faulting may require grinding or resurfacing.

The Georgia DOT has been conducting yearly pavement condition surveys of all jointed concrete pavement in the state for many years. The survey objectively rates roads to obtain an accurate record of the existing deterioration for each mile of pavement. The faulting at the joints is measured and visual distresses are tallied by the field survey crews. This data is then summarized in a yearly report. Pavement friction and roughness values are also included in this summary.

By knowing the rate and extent of deterioration, areas needing maintenance or rehabilitation can be determined. The data can help establish schedules for repair, estimate contract quantities and determine the effectiveness of rehabilitation procedures. Therefore, the survey needs to be as accurate as possible.

Conducting the Survey

The various pavement distresses are defined in the following sections along with descriptions and illustrations of the various levels of severity for each distress. The rater must be thoroughly familiar with the distresses and severity levels as defined in this section. The rater may or may not agree with all of the definitions and descriptions presented in this section, but all pavement sections must be rated in accordance with the criteria presented here in order for the rating system to be uniform.

Illustrations and photographs are shown for the various distresses and the severity levels which represent what typically might be seen in the field. The illustrations do not show all conditions that might be found nor is it intended that a condition must look exactly like what is shown in this manual for it to be rated at a particular severity level. The pictures are simple illustrations of what the rater is likely to see for a certain distress at a certain severity level. The rater must use his judgment based on the descriptions and the pictures while classifying the distress and severity level found on a sample section

This survey only addresses the structural condition of the pavement surface. It does not include skid resistance and ride-ability because these will be measured with high speed testing equipment.

The survey consists of measuring faulting of the joints and counting the occurrence of pavement defects in the outside lane for each mile of jointed concrete pavement in the state. The faulting of every eighth joint is measured using an electronic meter designed, developed and built by Office of Materials and Research personnel. The rest of the survey consists of a visual tally of horizontally broken slabs, longitudinal cracks, replaced slabs, spalled joints, patched joints, failed spall patches, and shoulder deterioration. Any special conditions such as grinding, undersealing, and so on should also be noted on the survey sheets. The visual tally will include every slab for each mile of concrete.

Faulting measurements and visual pavement distress will be tallied for whole miles on a milepost by milepost basis. Missing mileposts will require an estimated location. Data will be recorded for the outside lane and then placed on the Concrete Survey Form. The form can be found in the maintenance reference manual; it can be printed out and reproduced as many times as needed.

Safety

Before learning more about how to conduct the survey, let's discuss the most important consideration – Safety. The main thing to remember is this: Safety is the prime consideration. If an area can't be tested safely, skip it. Follow the current Maintenance Traffic Control Standards when performing the survey.

- This is a "rolling" operation and no traffic lane is to be blocked. The survey vehicle should be kept on the shoulder as far away from the mainline pavement as possible. This is important so that the fault meter operator can easily see around the vehicle while staying away from traffic.
- At no time will any vehicle move onto the mainline pavement during the survey operation, except at narrow bridges where special care should be taken. Of course, the survey should be discontinued and the vehicle crosses the bridge as soon as possible.
- Prior to placing the faulting gauge on the pavement edge for faulting measurement, the operator should look for a break in traffic. At no time should traffic be "waved" over or the faulting gauge be placed on the pavement in close proximity to an oncoming vehicle.
- Extra care must be taken when crossing ramp areas. Do not take any faulting measurements in the ramp or flares. Make visual observations only in the areas where the vehicles can stay safely on the shoulder. Try to stay out of the ramp gore area as much as possible. When coming to an exit ramp for instance, stop taking faulting measurements as soon as the flare starts. Do not walk across the flare to take readings. The visual survey can safely continue while the vehicle stays on the shoulder to a point up the ramp above the gore area.

- The entrance ramp is handled in a similar manner, except the steps are reversed. Resume visual survey at this point and faulting measurements at the end of the flare.

Faulting

Description and Possible Causes

Faulting is a difference in elevation across a joint or crack usually associated with undoweled JPCP. Usually the approach slab is higher than the leave slab due to pumping, the most common faulting mechanism. Faulting is noticeable when the average faulting in the pavement section reaches about 2.5 mm (0.1 inch). When the average faulting reaches 4 mm (0.15 in), diamond grinding or other rehabilitation measures should be considered (Rao et al., 1999). Most commonly, faulting is a result of slab pumping. Faulting can also be caused by slab settlement, curling and warping. It is primarily cause by traffic loadings.

Measurements

The faulting of every eighth joint is measured using an electronic meter built by Office of Materials and Research personnel. As each reading is taken it should be entered on the Concrete Survey Form. Be sure to include the minus sign if the meter reads -1, -2, and so on. The only exception is –0 since this is the same as zero. Use a comma to separate individual readings. There should be approximately 22 faulting readings per mile for 30 foot joint spacings and 33 for 20 foot spacings. Mainline bridges and ramps will reduce the number of readings. Enter the total number of tests made. The Electronic Fault meter was designed to simplify measuring joint faulting. It reads out directly in 32nds of an inch and shows whether the reading is positive or negative. The unit reads out in 1 second and freezes the reading in a display so it can be removed from the road before reading for a safer operation.

To operate the unit: (1) Although the meter is very stable, it should be checked at the beginning of each day and after lunch to assure correct readings. Set the meter on the cal. block with the front end lined up with the cal. 12 mark. If a reading of 12 is not obtained, initially, push the button several more times to insure stability. In this position the probe rests on a 3/8 inch block. Press the button, as 3/8 inch = 12/32, a reading of 12 is obtained. Set the meter to line up with the zero mark. Press the button. The meter should read zero. As long as the zero and 12 readings are obtained, the unit is working properly. If not, discontinue testing and call the OMR so the problem can be corrected. (2) Grip the handle of the meter with the thumb resting lightly on the test button. The operator should stand safely on the shoulder facing traffic while making the test. <u>There is an arrow on the meter showing traffic direction.</u> The faultmeter must be oriented correctly to get accurate readings. (3) Be sure to wait for a break in traffic before testing. Do not wave traffic over. (4) Set the meter on the leave side of the joint approximately 6 inches inside the white line. A probe contacts the slab on the approach slab. The joint must be centered between the two marks on each side of the meter. (5) Push, then instantly release the test button. A 1-second tone will sound. (6)

As soon as the tone stops, lift the meter and move away from the pavement. The meter will remain “frozen” until the next reading is taken. This allows the operator to move away from traffic before the meter is read.

To summarize: (1) Calibrate the meter at the beginning of each day and after lunch. (2) Wait for a break in traffic. (3) Sit the meter on the pavement approximately 6 inches inside the white line with the joint between the guide lines. (4) Push and release the test button. (5) As soon as the tone stops, move the meter from the pavement before reading. (6) For additional tests, repeat steps 2 through 5.

Should the batteries need replacing, the reading will still lock in the display after 1 second, but the 1 second continuous tone will be followed by a 3 second pulsating tone and “LO BATT” will show in the display. The readings will still be accurate but the batteries should be replaced within a day or so.

Broken Slabs

Description and Possible Causes

They are combination of corner break; cracks that intersect the PCC slab joint near the corner or panel cracking which extend across the entire and typically divide an individual slab into two to four pieces. They cause; roughness, moisture infiltration, faulting, spalling and disintegration. They are caused by load repetitions combined with a loss of support, poor load transfer across the joint or crack, curling stresses and warping stresses.

Measurements

Every broken slab in the outside lane of each mile will be visually counted. Surface cracks do not count; the slab must be in your opinion actually broken. There are two severity levels for broken slabs.

Severity level 1 - The broken slab has a hairline and “tight” working crack regardless of its length.

Severity level 2 - The broken slab is has a moving crack that may be wide, spalled and needs to be sealed; in your opinion, the slab is actually broken.

With these two severity levels we can get an accurate account of all slabs that may be cracked or broken and we can pin-point those slabs that require replacement. Once these locations are noted our maintenance forces can schedule rehabilitation work to prevent further deterioration. Never count the same type distress more than once per slab; if a slab has one break or several breaks, count it as one broken slab for whatever severity level it is. If a slab has both severity level breaks, count only the worst of the two severity levels (See Figure A-A). As the distresses are being counted place a single tick mark in the appropriate column on the survey form. When the form is completed, show the total number of broken slabs for each severity level.

FIGURE -1 BROKEN SLABS LEVEL 1

FIGURE-2 BROKEN SLABSLEVEL 2

Slabs with Longitudinal Cracks

Description and Possible Causes

These cracks are predominantly parallel to the pavement centerline. The possible conditions are; wide joint opening, corrosion of tie bar, poor load transfer, pumping, and spalled or deteriorated concrete pavement.

Measurement

Slabs will be visually inspected for the presence of longitudinal cracks. Longitudinal cracks begin at the joint, usually in the wheel paths, and with time, grow in length. Remember, slabs with longitudinal cracks are counted, not the number of longitudinal cracks. The same rule applies when counting the same type distresses.

The severity of the longitudinal cracks will be rated as follows:

Severity level 1 – The longitudinal crack is a hairline and "tight" working crack.

Severity level 2 – The longitudinal crack is a moving crack generally wider and may be spalled, and needs to be sealed.

The highest severity longitudinal crack on a slab is counted if both severity levels are found on one slab. Also, if a longitudinal crack is present on both sides of the joint each slab is counted as having a longitudinal crack. (See Figure B-B). If a slab has three severity level 1 longitudinal cracks and one level 2 longitudinal crack, the slab is counted as one slab with severity level 2 longitudinal cracks.

FIGURE-3 SLABS WITH LONGITUDINAL CRACKS LEVEL 1

FIGURE -4 SLABS WITH LONGITUDINAL CRACKS LEVEL 2

Replaced Slabs

Description and Possible Causes

These are full depth patched or patched slabs resulting among other; from corner breaks, faulting, joint transfer system deterioration, and blowup.

Measurements

A count of all slabs with replacements will be made within each mile. Some replacements are not so obvious because the color and texture are similar. This is especially true after the pavement has been ground for some time.

All replaced slabs will be counted in each mile as they occur. Also, if a replaced slab has cracked, it counts as a failed replaced slab and will be noted on the survey form as such but when inputting the data into the survey program it will be counted as a broken slab. If there is a replaced slab and a broken slab within the same 20 foot or 30 foot section count both the replaced slab and the broken slab (See Figure C-C). Furthermore, if a replacement is on both sides of the original joint each slab replacement is counted (See Figure D-D). When the survey form is completed show the total replaced slabs. While there aren't any deduction points for replaced slabs it is good to get an account of how many slabs have been replaced for information purposes.

FIGURE – 5 REPLACED SLABS

Failed Replaced Slab

Description and Possible Causes

These are distresses on newly replaced slabs. They are caused by load repetitions combined with a loss of support.

Measurements

A count of all failed replaced slabs will be made. By knowing how many replacements we have that has failed provides us with the necessary information on how the materials we use are performing. Failed replaced slabs will be counted as such on the survey form but at the data entry phase they will be counted as broken slabs since they will have to be replaced again.

FIGURE – 6 FAILED REPLACED SLAB

Joint Defects

Description and Possible Causes

These distresses are breaking or spalls of joint edges. Usually occurs within about 0.6 m (2 ft.) of joint edge. Possible causes are; excessive stresses at the joint caused by infiltration of incompressible materials and subsequent expansion (can also cause blowups), disintegration of the PCC from freeze-thaw action or "D" cracking, weak PCC at a joint caused by inadequate consolidation during construction. This can sometimes occur at a construction joint if (1) low quality PCC is used to fill in the last bit of slab volume or (2) dowels are improperly inserted, misalignment or corroded dowel, and heavy traffic loadings. They cause debris on the pavement, roughness which is generally an indicator of joint deterioration.

Measurements

There are three types of joint defects visually counted for this survey:

- Joints with Spalls: The number or severity of spalls is not accounted for, just the total number of joints with spalls; therefore, if a joint has two or more spalls a single “tick” mark is made showing one spalled joint. According to the specifications, a spall must be at least 1.5” x 6” in area. We will use this minimum size to define the minimum spall size.
- Joints with patched spalls: Only well performing spall patches are counted. Spalls patched with asphalt should be counted as spalls rather than patched spalls because the asphalt patched is only a temporary repair until the area can be sawed out and patched with concrete.

FIGURE – 7 JOINTS WITH SPALLS

FIGURE – 8 JOINTS WITH FAILED SPALL PATCHES

- Joints with failed spall patches: The patched spall has cracked or the patching material has come out and will have to be replaced. These are placed on the form as a failed spall patch. Failed spall patches provide us with information on how the materials use is performing.

Every joint in each mile is to be visually checked for joint defects. We are looking for the total number of joints with defects not a count of individual joint defects. To illustrate, a joint with one or more spalls, or failed spall patches would count as one spalled joint and noted on the survey form (See figure E-E).

FIGURE – 9 JOINTS WITH FAILED SPALL PATCHES

Shoulder Joint Distress

Description and possible Causes

They are lane-to-shoulder Drop-off, lane-to-shoulder separation and patch deterioration along the shoulder. This distress is caused by slab movement under load and usually occurs at the joints. The distress takes the form of a depressed pot hole at the joint. This distress can advance to the extent of large holes at the joints and base material pumped out onto the shoulder.

Measurements

The shoulder joint will be visually inspected for distress.

There are two severity levels for shoulder joint distress:

Severity level 1 – Obvious depressions adjacent to transverse joints. Depressions are not large enough to require patching. No “pumping” of base material onto the shoulder is present.

FIGURE – 10 SHOULDER JOINT DISTRESS LEVEL 1

Severity level 2 – Large, deep depressions adjacent to transverse joints. Depressions are large enough to require patching. The “pumping” of base material onto the shoulders should be rated severity level 2 regardless of depression size.

The percentage of the mile affected by shoulder joint distress will be estimated and noted on the survey form (See Figure F-F). There can be a situation where both severity levels are present within a given mile. If this occurs just note the percentages of affected area for both severity levels in their appropriate column.

Once all the information as been tabulated on the survey form it shall be entered into the Data Program by the Bridge Manager so a rating can be generated.

Level 2

FIGURE – 11 SHOULDER JOINT DISTRESS LEVEL 2

F.2 OTHER DISTRESSES NOT COVERED BY CPACES

There are other types of defects which are not being considered in CPACES either because they occur infrequently or they are included in one of the above categories at a certain severity level.

They are included in this section to provide a general pavement survey overview. They include; Punchout, polish aggregate and Scalling, D cracking, Map cracking, blow-ups, water bleeding and pumping and pop-outs.

Punch-out

Punch-outs are localized areas of distress that occur on CRCP. They are characterized by a failed rectangular section of concrete that is enclosed by (1) two closely spaced transverse cracks, (2) a short intersecting longitudinal crack, and (3) the outside pavement edge. Most punch-outs occur on the outside pavement edge, although some punch-outs occur adjacent to the longitudinal joint. Punch-outs occur at locations where two closely spaced (typically less than 0.6 m apart) transverse cracks exist, where support beneath the cracks (particularly at the outside edges) has been reduced due to pumping and erosion, and where aggregate interlock across the cracks has diminished. Concrete properties that influence the development of punch-outs include elastic modulus, strength, drying shrinkage, and coefficient of thermal expansion. Related aggregate properties include the elastic modulus, strength, and coefficient of thermal expansion. Other aggregate factors that influence crack spacing and aggregate interlock include the aggregate gradation, size, shape, angularity, texture, and abrasion resistance.

FIGURE – 12 PUNCHOUT

Scaling

Scaling is the deterioration of the upper concrete slab surface, normally 3 mm to 13 mm, and may occur anywhere over the pavement.

FIGURE – 13 SCALING

D Cracking

D-cracking, or durability cracking, appears as a series of closely spaced, crescent-shaped cracks along joints or cracks. The cracking and accompanying staining (from calcium hydroxide or calcium carbonate residue) often appear in an hourglass shape on the pavement surface. D-cracking occurs when water in certain aggregates freezes, leading to expansion and cracking of the aggregate and/or surrounding mortar. The rapid expulsion of water from the aggregates may also contribute to dissolution of soluble paste components (Van Dam et al., 2002a). D-cracking deterioration often begins at the bottom of the slab where free moisture is available. Generally, it takes 10 to 20 years (sometimes more) for D-cracking to develop depending on the aggregate type and pore structure, climatic factors, availability of moisture, and concrete properties. The coarse aggregate type plays a role in the development of D-cracking. Most D-cracking-susceptible aggregates are of sedimentary origin commonly composed of limestone, dolomite, or chert (Stark, 1976). Key aggregate properties related to D-cracking susceptibility are mineralogy, pore structure, absorption, and size (Schwartz, 1987). To mitigate the development of D-cracking, many Midwestern states have specified a smaller maximum aggregate size, although this reduction often results in concrete mixtures with greater shrinkage and reduced aggregate interlock capabilities. The presence of deicing salts exacerbates the potential for D-cracking for certain carbonate aggregates (Dubberke and Marks, 1985).

FIGURE - 14 D CRACKING

Map Cracking

Map Cracking are Intersecting cracks that extend below the surface of hardened concrete; caused by shrinkage of the drying concrete surface which is restrained by concrete at greater depths where either little or no shrinkage occurs; vary in width from fine and barely visible to open and well-defined. The chief symptom of chemical reaction between alkalis in cement and mineral constituents in aggregate within hardened concrete; due to differential rate of volume change in different portions of the concrete; cracking is usually random and on a fairly large scale, and in severe instances the cracks may reach a width of 0.50 in.

FIGURE – 15 MAP CRACKING

Blowups

Blowups are localized upward movements that occur at joints and cracks and are often accompanied by shattered or fragmented concrete in that area. Blowups result from excessive expansive pressures that occur in the pavement because of intrusion of incompressibles into joints and cracks, presence of reactive aggregates, or extremely high pavement temperatures and moisture conditions (Hoerner et al., 2001). Blowups generally take several years to develop and occur when the pavement can no longer accommodate continued slab expansions. They are more commonly associated with JRCP designs constructed with long joint spacing and, therefore, experience large slab movements. Deteriorated joints or the presence of D-cracking may also contribute to the development of blowups. The development of blowups is largely influenced by the coefficient of thermal expansion of the concrete that is strongly related to the coefficient of thermal expansion of the coarse aggregate. Other aggregate properties affecting blowups include aggregate mineralogy and elastic modulus.

FIGURE - 16 BLOWUPS

Water Pumping and Bleeding

Pumping is the ejection of material by water through joints or cracks, caused by deflection of the slab under moving loads. As the water is ejected, it carries particles of gravel, sand, clay or silt, resulting in a progressive loss of support. Surface staining or accumulation of base or subgrade material on the pavement surface close to joints or cracks is evidence of pumping. Water bleeding occurs when water seeps out of joints or cracks.

FIGURE - 17 WATER PUMPING AND BLEEDING

Popouts

Popouts are defined as small pieces of pavement broken loose from the surface, normally ranging in diameter from 25 mm to 100 mm and depth from 13 mm to 50 mm.

FIGURE – 18 POPOUTS

Alkali-Silica Reactivity (ASR)

ASR results from the reaction of the alkalis in the cement with the siliceous components of certain aggregates. This reaction produces a gel that significantly expands in the presence of moisture, causing cracking of the surrounding cement matrix and the development of an irregular, map-like cracking, generally less than 50 mm deep, most often over the entire slab area. However, ASR can also lead to internal horizontal cracks at greater depths within the slab. With continued advancement, large pieces of concrete may dislodge from the center portions of the slab, and joint spalling, blowups, and other pressure-related distresses may also occur. A handbook depicting ASR distress in pavements and highway structures is available to aid in identifying ASR distress (Stark, D., *Handbook for the Identification of Alkali–Silica Reactivity in Highway Structures*, SHRP-C/FR-91-101, Strategic Highway Research Program, Washington, DC, 1991).

Appendix G Typical Soil Support Values

APPENDIX G

AVERAGE TYPICAL SOIL SUPPORT VALUES FOR ESTIMATING PURPOSES ONLY

DISTRICT 1

COUNTY	SOIL SUPP.
BANKS	3.0
BARROW	2.5
CLARKE	3.0
DAWSON	2.5
ELBERT	3.0
FORSYTH	3.0
FRANKLIN	3.0
GWINNETT	2.5
HABERSHAM	2.5
HALL	2.5
HART	2.5
JACKSON	3.0
LUMPKIN	2.5
MADISON	2.5
OCONEE	3.0
RABUN	3.0
STEPHENS	2.5
TOWNS	2.5
UNION	2.5
WALTON	3.0
WHITE	2.5

DISTRICT 2

COUNTY	SOIL SUPP.
BALDWIN	3.0
BLECKLEY	3.0
BURKE	3.0
COLUMBIA	3.0
DODGE	3.5
EMANUEL	3.5
GLASCOCK	3.0
GREENE	3.0
HANCOCK	3.0
JASPER	3.0
JEFFERSON	3.0
JENKINS	3.5
JOHNSON	3.5
LAURENS	3.5
LINCOLN	3.0
MCDUFFIE	3.0
MORGAN	2.5
NEWTON	2.5
OGLETHORPE	3.0
PUTNAM	3.0
RICHMOND	3.5
SCREVEN	3.5
TALIAFERRO	3.0
TREUTLEN	4.0
WARREN	3.0
WASHINGTON	3.0
WILKES	3.0
WILKINSON	3.0

DISTRICT 3

COUNTY	SOIL SUPP.
BIBB	3.0
BUTTS	3.0
CHATTAHOOCHEE	3.0
COWETA	2.5
CRAWFORD	3.0
DOOLY	3.5
FAYETTE	2.5
HARRIS	2.5
HEARD	2.5
HENRY	2.5
HOUSTON	3.0
JONES	2.5
LAMAR	3.0
MACON	3.5
MARION	3.0
MERIWETHER	3.0
MONROE	2.5
MUSCOGEE	2.5
PEACH	3.5
PIKE	3.0
PULASKI	3.5
SCHLEY	3.0
SPALDING	2.5
STEWART	3.5
SUMTER	3.5
TALBOT	3.0
TAYLOR	3.0
TROUP	3.0
TWIGGS	2.5
UPSON	3.0
WEBSTER	3.5

DISTRICT 4

COUNTY	SOIL SUPP.
ATKINSON	4.0
BAKER	4.0
BEN HILL	4.0
BERRIEN	4.0
BROOKS	4.0
CALHOUN	3.5
CLAY	3.5
CLINCH	4.0
COFFEE	4.0
COLQUITT	3.5
COOK	4.0
CRISP	4.0
DECATUR	4.0
DOUGHERTY	3.5
EARLY	3.5
ECHOLS	4.5
GRADY	4.0
IRWIN	4.0
LANIER	4.0
LEE	3.5
LOWNDES	4.0
MILLER	3.5
MITCHEL	3.5
QUITMAN	3.5
RANDOLPH	3.5
SEMINOLE	4.0
TERRELL	3.5
THOMAS	4.0
TIFT	4.0
TURNER	4.0
WILCOX	3.5
WORTH	3.5

DISTRICT 5	
COUNTY	SOIL SUPP.
APPLING	4.0
BACON	4.0
BRANTLEY	4.0
BRYAN	4.0
BULLOCH	4.0
CAMDEN	4.0
CANDLER	4.0
CHARLTON	4.0
CHATHAM	4.0
EFFINGHAM	4.0
EVANS	4.0
GLYNN	4.0
JEFF DAVIS	4.0
LIBERTY	4.0
LONG	4.0
MCINTOSH	4.0
MONTGOMERY	4.0
PIERCE	4.0
TATTNALL	4.0
TELFAIR	4.0
TOOMBS	4.0
WARE	4.0
WAYNE	4.0
WHEELER	4.0

DISTRICT 6	
COUNTY	SOIL SUPP.
BARTOW	2.5
CARROLL	2.5
CATOOSA	2.0
CHATTOOGA	2.5
CHEROKEE	2.5
DADE	2.5
FANNIN	2.5
FLOYD	2.5
GILMER	2.5
GORDON	2.0
HARALSON	2.5
MURRAY	2.5
PAULDING	2.5
PICKENS	2.5
POLK	2.5
WALKER	2.5
WHITFIELD	2.0

DISTRICT 7	
COUNTY	SOIL SUPP.
CLAYTON	2.5
COBB	2.0
DEKALB	2.0
DOUGLAS	2.5
FULTON	2.0
ROCKDALE	2.5

REVISED: May 27, 2005

Appendix H Regional Factors

REGIONAL FACTORS FOR USE IN FLEXIBLE PAVEMENT

FIGURE H-1

Appendix I Allowable Base Material Maps

APPENDIX I-1

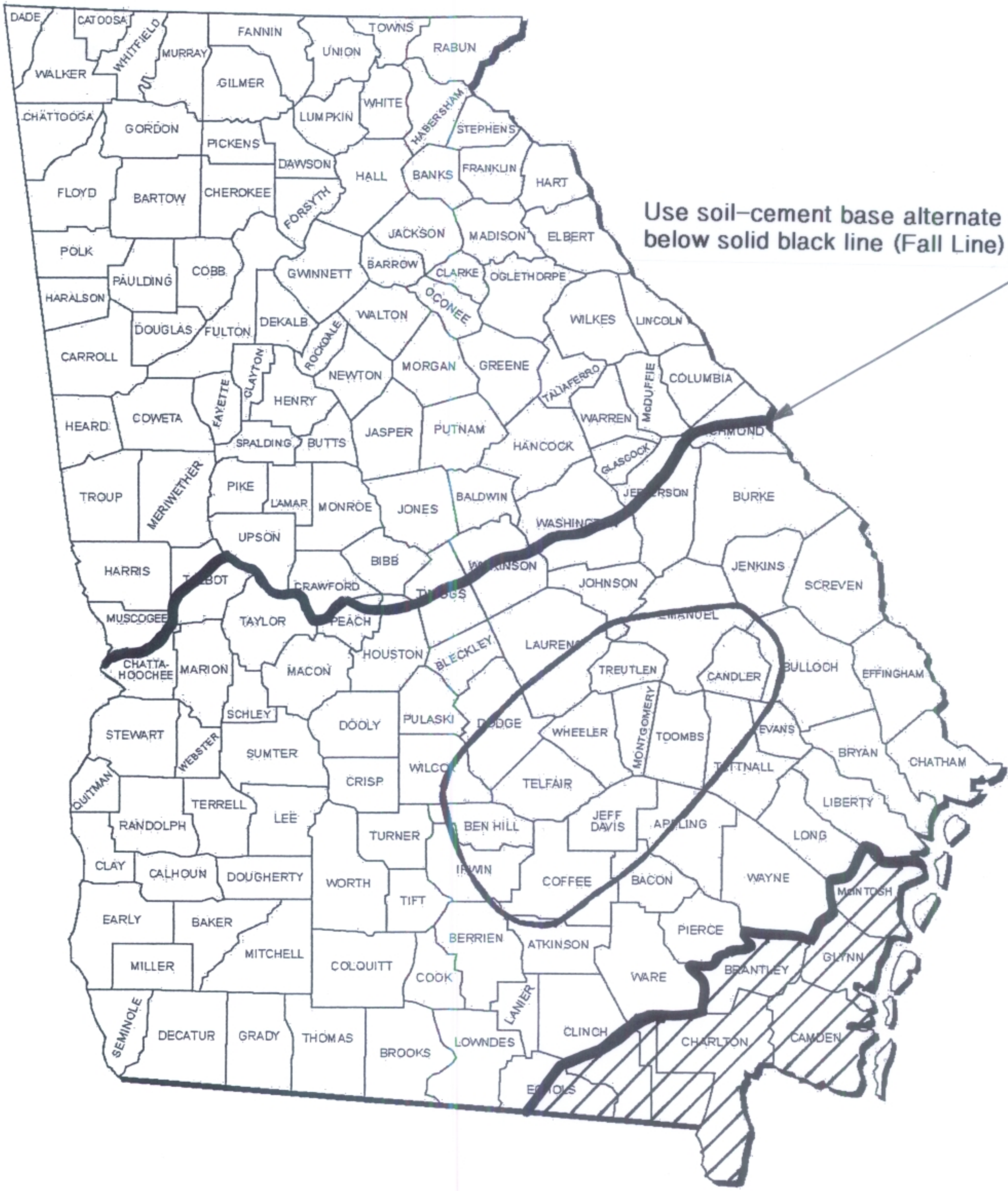
GAB is acceptable throughout Georgia excluding the
area inside the heavy line

APPENDIX I-2

Soil Cement is acceptable south of the heavy line

APPENDIX I-3

AC Base is acceptable south of heavy line



GAB is acceptable throughout Georgia. It may not be economically feasible to haul GAB to the counties within the bold line.

Do not use soil-cement in the hatched area because of gap-graded sands.

Appendix J: Rigid Pavement Design Analysis

RIGID PAVEMENT DESIGN ANALYSIS
(BASED ON AASHO INTERIM GUIDE FOR THE DESIGN OF RIGID PAVEMENT STRUCTURES)

P.I. NO.: **PROJECT NUMBER:** **COUNTY:**
LENGTH: **TYPE SECTION:**
DESCRIPTION:
TYPE OF ADJOINING PAVEMENT: **BEGINNING OF PROJECT:**
END OF PROJECT:

TRAFFIC DATA: **24 HR. TRUCK PERCENTAGE:**

ONE-WAY AADT BEGINNING OF DESIGN PERIOD: **VPD** **YEAR**
ONE-WAY AADT END OF DESIGN PERIOD: **VPD** **YEAR**
MEAN AADT (ONE WAY): **VPD**

DESIGN LOADING:
DESIGN LANE TRAFFIC

MEAN AADT		LDF		TRUCKS		18K ESAL		
	X		X	MU	X	2.68	=	
	X		X	SU	X	0.5	=	
	X		X	Other	X	0.004	=	
					TOTAL DAILY LOADING		=	

TOTAL DESIGN PERIOD LOADING = (loads/day)*(20 years)*(365 days/year) = total loads

DESIGN DATA: **SERVICEABILITY (Pt):** **WORKING STRESS:**
SOIL SUPPORT VALUE:
MODULUS OF SUBGRADE REACTION K =
MODULUS OF SUBBASE REACTION K_1 =
TRIAL DEPTH OF CONCRETE PAVEMENT

ACTUAL STRESS FROM NOMOGRAPH:
PERCENT OVER-UNDER DESIGN: % understressed
% overdesigned

RECOMMENDED RIGID PAVEMENT STRUCTURE:

REMARKS:

PREPARED BY:

RECOMMENDED: ______________________________

STATE URBAN DESIGN ENGINEER **DATE**

APPROVED: ______________________________

STATE PAVEMENT ENGINEER **DATE**